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SAN ANIONIO RIVER AND TRIBUTARIES, TEXAS

FOUNDATION REPORT

SAN PEDRO CREEK TUNNEL AND SHAFTS

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DEPARTMENT OF THE ARMY FORT WORTH DISTRICT, CORPS OF ENGINEERS FORT WORTH, TEXAS

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FOUNDATION REPORT

San Pedro Creek Tunnel and Shafts

Statement A per telecon Vichy Sharp Army Ft Worth District Corps of Eng. ATIN: CESWF-IM-C Fort Worth, TX 76102-0300 NWW 9/4/91

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FOUNDATION REPORT SAN PEDRO CREEK TUNNEL AND SHAFTS

TABLE OF CONTENTS

Paragraph	<u>Title</u>	Page		
PART I: INTRODUCTION				
1-01 1-02 1-03 1-04 1-05	Location and Description of Project. Construction Authority Purpose of Report Contractor and Contract Supervision Disputes Review Board References	1 1 2 2 2 2 3		
	PART II: FOUNDATION EXPLORATIONS			
2-01 2-02	Investigation Prior to Construction Investigations During Consruction	5 6		
	PART III: GEOLOGY			
3-01	Regional Geology a. Physiography b. Stratigraphy c. Structure	7 7 7		
3-02	Geology of the Tunnel Alignment a. Overburden b. Primary Formation c. Geologic Structure d. Formation Weathering e. Ground Water f. Seismicity g. Engineering Characteristics of Overburden h. Engineering Characteristics of Primary Formation	8 9 11 11 14 14 14 15		
	PART IV: SPECIAL DESIGN CONSIDERATIONS			
4-01 4-02 4-03	Construction Method Swell Pressures Heave Potential	17 18 19		

1ABLE OF CONTENTS (CONT)

PART V: EXCAVATION AND SUPPORT PROCEDURES

5-01	General .	21
5-02	Excavation Equipment	
	a. Shaft Excavation Equipment	21
	b. Tunnel Boring Machine (TBM)	22
5-03	Pre-cast Tunnel Liner	23
5-04	Foundation Preparation	24
5-05	Outlet Shaft Excavation	25
5-06	Inlet Shaft Excavaion	26
5-07	Maintenance Shaft Excavation	29
5-08	Vent Shaft Excavations	30
5-09	Hydraulic Instrumentation Shaft Excavations	30
5-10	Tunnel Excavation	31
	PART VI: CHARACTER OF FOUNDATION OR TUNNELING MED	NUI
6-01	General	33
6-02	Tunnel Foundation or Medium	33
6-03	Outlet Shaft Foundation	35
6-04	Inlet Shaft Foundation	35
6-05	Maintenance Shaft Foundation	36
6-06	Vent Shaft Foundations	37
6-07	Hydraulic Instrumentation Shaft Foundations	38
	PART VII. FOUNDATION TREATMENT	
7-01	General	39
7-02	Rock Anchors	39
7-03	Tunnel Liner Grouting	41
	PART VIII - CONSTRUCTION MATERIALS	43
	PART IX: GEOTECHNICAL INSTRUMENTATION	
9-01	General	45
9-02	Outlet Shaft Instrumentation	45
9-03	Tunnel Instrumentation	46
	PART X: FOUNDATION PROBLEM AREAS	49
	PART XI. RECORD OF FOUNDATION INSPECTIONS AND GEOLOGIC DOCUMENTATION	J1

TABLE OF CONTENTS (CONT)

APPENDIX A Photographs

Ĺ

APPENDIX R Tunneling Progress Charts

APPENDIX C Tunnel Liner Grouting Data

APPENDIX D Drilled Shaft Logs

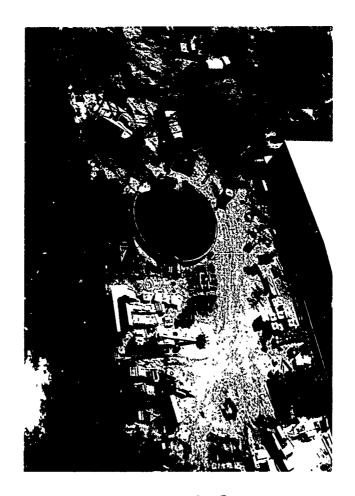
APPENDIX E Plates (maps)

APPENDIX F Boring Logs (design investigations)

APPENDIX G Geotechnical Instrumentation Report



SAN PEDRO CREEK INLET SHAFT Aerial View Southeast



SAN PEDRO CREEK
OUTLET SHAFT
Aerial View
Southwest

PART I

INTRODUCTION

1-01. Location and Description of Project. This project, "The San Antonio River and San Pedro Creek Tunnels, Phase II-Tunnels and Shafts," is part of the broader San Antonio Channel Improvement Project. The latter is a flood control project for the upper San Antonio River and four tributaries.—Martinez, Alazan, Apache, and San Pedro Creeks. The subject of this report is a tunnel constructed on San Pedro Creek.

San Pedro Creek Tunnel is the shorter of the two inverted siphon tunnels which have been designed to prevent flooding in downtown San Antonio, Texas. Both tunnels are of the same design and same general dimensions, and have been excavated by the same tunnel boring machine (TBM). Each tunnel will divert flood waters from its respective drainage into an inlet shaft located upstream from the city, and transfer the water beneath the city to an outlet shaft downstream. San Pedro Creek Tunnel extends 5,985 feet from the center of the inlet shaft to the center of the outlet shaft. The longer San Antonio River Tunnel, the subject of a later report, extends 16,225 feet between the centers of its inlet and outlet shafts.

The subject tunnel follows the course of San Pedro Creek in an easterly arc between Interstate Highway 35 on the north and Guadalupe Street on the south. The tunnel slopes downstream at a gradient of .002 from an invert depth of 117 feet (elev. 506) at the inlet to 145 feet (elev. 494) at the outlet. The lining is 12-inch thick precast concrete which gives an inside tunnel diameter of 24 feet 4 inches.

There are seven shafts along San Pedro Creek Tunnel. The inlet shaft is located just south of the intersection of Interstate Highways 35 and 10, and lies between Santa Rosa Street on the west and Camaron Street on the east. It has a cast-in-place concrete liner with a I.D. of 24 feet 4 inches. An 18-foot I.D. cast-in-place concrete maintenance shaft is located approximately 100 feet south of the Travis Street Bridge and just west of Cameron Street. Two 4-foot I.D. steel pipe ventilation shafts are located respectively about 100 feet south of Salinas Street and about 100 feet north of Durango Street. Two 12-inch I.D. steel pipe shafts are located respectively within approximately 150 feet of the inlet shaft and the outlet shaft; these shafts facilitate hydraulic instrumentation measurements once the tunnel is in operation. The outlet shaft is located about 130 feet north of Guadalupe Street just west of San Pedro Creek; it is lined with cast-in-place concrete to an I.D. of 35 feet.

1-02. <u>Construction Authority.</u> Construction of the San Antonio Channel Improvement Project was authorized in the Flood Control Act of 1954 which was approved on September 3, 1954 (Public Law 780, 83rd Congress, 2nd Session).

- 1-03. <u>Purpose of Report.</u> The objective of this report is to describe the foundation conditions encountered during the construction of the subject tunnel and shafts. It is also intended to be a consolidated record of the foundation related construction operations and an information source for future reference. The report is to be a part of the permanent project engineering and construction record, and will provide background knowledge for evaluation of any future structural problems or further foundation studies.
- 1-04. Contractor and Contract Supervision. Obbayashi Corporation of Tokyo, Japan and San Francisco, California was awarded construction of the "San Antonio River and San Pedro Creek Tunnels, Phase II -Tunnels and Shafts" under Contract No. DACW63-87-C-0109 on September 23, 1987. The contract amount was \$47,750,000.40. The Notice to Proceed was issued on October 30, 1987, and the contractor acknowledged receipt on November 3, 1987.

Subcontractors to Ohbayashi on the San Pedro Creek Tunnel included Boretec Inc. of Solon, Ohio who selected and re-manufactured a used TBM for the job; Schulster Company Inc., of Milwaukee, Wisconsin, who manufactured the precast concrete liner segments at a plant established in San Antonio; Woodward-Clyde Consultants of Houston, Texas who were responsible for the specified geotechnical instrumentation program; Cato Electric and Drilling of San Antonio who constructed the concrete soldier piers for the maintenance shaft, Beck Foundation Company of San Antonio who drilled the maintenance, vent, and hydraulic instrumentation shafts, and J-Mar Construction who contracted the muck hauling.

Quality control was provided by the principal contractor, Ohbayashi Corporation. The contractor was required to establish and maintain an effective quality control system consisting of plans, procedures, and organization to insure the contract requirements in materials, equipment, workmanship, fabrication, and construction operations. A quality control system manager (Mr. Lindy White) from within the contractor's organization was required to be at the worksite with responsibility for regulating all quality control matters. A fully qualified staff was required under the system manager with necessary experience and technical training to perform all quality control activities. Records and tests of the contractor's quality control throughout the construction operations were furnished to the Government, as directed by the Contracting Officer. The entire work was subject to inspection and testing by the Government as quality assurance prior to acceptance.

Ohbayashi Corporation's contract supervision was provided by Mr. Kaname Tonoda, General Manager in the San Francisco Office, Mr Carl Linden, on-site Project Sponsor, and Mr. Paul Zick, on-site Project Manager.

The Government's contract administration and quality assurance was provided under Col. William D Brown, the Contracting Officer. Mr Keith M. Allen was the Resident Engineer and Authorized Representative of the Contracting Officer.

1-05 <u>Disputes Review Board.</u> The Disputes Review Board was an advisory body created by mutual agreement between the Government and Ohbayashi Corporation

to assist in the resolution of disputes or claims arising out of the project. The process was a voluntary, expedited and non-judicial, non-binding mediation procedure, whereby an independent three-party Board was presented with Government-Contractor disputes for expert evaluation, recommendations, and possible resolution.

The Board consisted of one member selected by the Government, Mr. Ronald E. Heuer, one member selected by Ohbayashi, Mr. P.E. Sperry, and the final member, Mr. Robert J. Smith, who was selected by the first two members.

The Government and the Contractor were required to mutually agree to submit a dispute to the Disputes Review Board, and the Board's resulting recommendations were non-binding to either party. If the dispute remained unresolved after 30 days following the receipt of the Board's recommendations, the Contractor could submit a request for a Contracting Officer's Decision under the "Disputes" clause of the contract.

This report was prepared by the Resident Geologist, Mr. Roy Crutchfield, during construction of the subject tunnel. The Resident Engineer was Mr. Keith Allen who succeeded Mr. Bob Wortham in November 1988. The Chief of Construction Division was Mr. Shigeru Fujiwara. The Fort Worth District Engineer was Colonel John Schaufelberger succeeded by Colonel William Brown in September 1989.

Consultation and support in preparation of the report was provided by the Fort Worth District Geotechnical Branch, Engineering Division. Mr. Mel Green was Chief of Geotechnical Branch, Mr. Bob Behm was Chief of Engineering Geology Section, and Mr. Harlan Karbs was Chief of the Soils Design Section.

1-06. References.

- a. Design Summary Report with Appendices A and $B_{\rm g}$ San Antonio River and San Pedro Creek Tunnels, Phase II -Tunnels and Shafts, Solicitation No. DACW63-87-B-0085, dated May 1987.
- b. Design Memorandum No. 5, Part III, Supplement I, Construction Unit 7-3-1, dated November 1985.
- c. Geologic Atlas of Texas, San Antonio Sheet, Project Director Virgil E. Barnes, Univ of Texas at Austin, Bureau of Economic Geology, 1983 revised edition.
- d. A Revision of Taylor Nomenclature, Upper Cretaceous, Central Texas by Keith Young, Bureau of Economic Geology, Geological Circular 65-3, dated May 1965
- e. Ground-Water Geology of Bexar County, Texas by Ted Arnow, Geological Survey Water-Supply Paper 1588, dated 1963.
- f. Geologic Map of Bexar County, Texas by A N. Sayre, dated 1932-33 (with modifications by Lang, Brown, Mitchell, and Arnow dated 1959).

g. The Geology of Texas, Volume I, Stratigraphy by Sellards, Adkins, and Plummer. The University of Texas Bulletin No. 3232, dated August 1932.

PART II

FOUNDATION EXPLORATIONS

2-01. <u>Investigation Prior to Construction</u>. Subsurface investigations prior to tunneling consisted of 25 borings drilled in five phases as the channel improvement plan for this reach of San Pedro Creek developed. The borings ranged from 23-foot deep auger holes to 180-foot deep core holes, and provided 3,085.9 linear feet of drilling exploration. Overburden was usually drilled with 6 to 8-inch augers except when undisturbed samples were taken with 6-inch Denison Barrel or 4-inch Shelby Tube. The primary for: ation was drilled with fishtail bits or 4 to 6-inch core berrels. All of the borings were drilled under supervision of the Corps of Engineers' Fort Worth District Office with either a Corps drill rig and crew or by contract driller with a Corps geologist. All on-site material evaluation, logging, and photographing was performed by a Corps geologist. Electric logs including resistivity, gamma, and caliper were obtained on the deeper borings, however, due to malfunctions of the resistivity equipment, the gamma logs proved most reliable and consistent for strata correlations.

The first five borings were drilled along the tunnel alignment in June 1975 and May 1981 as shallow investigations for flood capacity improvements planned for the San Pedro Creek channel. This plan was replaced by the tunnel project, but the borings provided relevant near-surface information. These first five borings were 8A-223, 6DC-235, 6DC-236, 6DC-237, and 6DC-238 which extended to respective depths of 23.0, 55 3, 52.0, 48.0, and 51.5 feet

In March and May of 1984, the first tunnel alignment, which was a straight course between the present inlet and a planned outlet near Durango Street, was explored with six borings. Boring 6DC-279 at the inlet and Boring 6DC-287 at the outlet obtained undisturbed samples through the overburden with a 6-inch Denison Barrel and took 6-inch continuous core samples through the primary formation. The remaining four borings, 6A4C-280, 6A4C-281, 6A4C-282, and 6A4C-285, were core sampled only below elevation 540, generally from 20 feet above the tunnel crown to 20 feet below the invert. The material above elevation 540 was drilled with rockbits in primary strata and predominantly with augers in overburden. The only exception to this procedure was that Boring 6A4C-285 was drilled with a fishtail bit in the bottom 10 feet

The initial straight alignment was abandoned in mid 1984 in favor of a shorter version of the current alignment which underlies the curved meander of San Pedro Creek The maximum separation of the two alignments was only about 750 feet; therefore, it was decided that the new alignment could be evaluated through electric log correlations between fishtail borings rather than obtaining additional core samples. Consequently, Borings 3F-283, 3F-284, 3F-295, AND 3F-296 were drilled with 8 and 10-inch augers to a depth of about 40 feet followed by 5 7/8-inch fishtail bits to total depths of 180 feet, approximately 20-feet below invert elevation. One additional core boring, 6A4C-286, was drilled about midway along the new alignment. It was augured to a depth of 51.5 feet and then cored with a 5 1/2-inch core barrel

to a depth of 180 feet. All five of these borings were drilled in August and September 1984.

In 1985, the tunnel alignment was extended 1,718 feet downstream to obtain an outlet site which could provide a larger staging area for construction. This was the present outlet site which is just north of Guadalupe Street and adjacent to San Pedro Creek. Therefore, the final alignment was established, and four borings were added in 1985 and 1986 to complete design investigations for the tunnel and shafts. Boring 6DC-302 was drilled at the final outlet shaft location. Overburden for 6DC-302 was augured in the upper 4.5 feet, drilled with 6-inch Denison Barrel from 4.5 feet to 22.5 feet, and augured again from 22.5 feet to just within weathered primary material at the 31.5 depth. The primary formation was then cored with a 6-inch barrel to a depth of 180 feet. Boring 6A4C-303, located midway on the alignment extension, was augured to 23.0 feet and cored with a 4-inch barrel to 180 feet. Boring 6A4C-304, at Nueva Street, was augured to 40.5 feet, fishtailed from 40.5 to 100.0 feet and cored with 5 1/2-inch barrel to the total depth of 165.0 feet. Boring 6A4C-305, at Martin Street, was augured to 40.8 feet, followed by rockbit to a depth of 100 feet, and then 4-inch cored to 165.0 feet.

Finally, five additional borings were drilled at the inlet and outlet sites in April 1986. These were shallow investigations primarily for design of the inlet and outlet surface structures. The borings were 6D4C-306, 6D4C-307, 6D4C-308, 4S4C-314, and 4S4C-315 which had respective depths of 31.0, 28.0, 33.0, 54.5, and 49.0 feet. (Note: Letter designations in boring numbers represent method of drilling and sampling as follows: A - auger, C - core barrel, D - Denison Barrel, F - fishtail bit, S - Shelby Tube. Numbers preceding these letters indicate the diameter of boring. Logs of design borings are in Appendix F.)

2-02. <u>Investigations During Construction</u>. There was no exploratory drilling during construction, although some additional core sampling was required as part of the geotechnical instrumentation program.

Core samples were taken in Borings X-1 and X-2. These borings were drilled for the installation of vertical 6-position extensometers above the tunne! at Stations 143+75 and 158+47 respectively. Core from both borings confirmed that the primary formation at those stations was massive, unfractured, calcareous clay shale. Boring X-1 was augured 12 feet into unweathered shale to a depth of 48 feet, and then NX size (2.155 inches dia) core was taken to the total depth (logs in Appendix G) NX core samples were also taken in 7 borescope observation holes drilled in the tunnel walls at each of the following stations: 143+63, 143+71, 143+79, 143+87, 143+95, 158+39, 158+47, and 158+55. This was a total of 56 borings drilled to an approximate depth of 8 feet. Each group of 7 borings had a 45 degree spacing around the tunnel circumference, starting at 45 degrees from the invert centerline; no boring was drilled in the invert. The material was massive, calcareous clay shale with occasional fracturing, due primarily to stress relief around the tunnel excavation.

PART III

CENTACY

3-01. Regional Geology.

- a. Physiography. The San Pedro Creek Tunnel is located where the northeast trending Balcones fault zone forms the boundary between two physiographic provinces, the Edwards Plateau to the northwest and the Gulf Coastal Plain to the southeast. The Edwards Plateau is located on the upthrown side of the fault zone with an altitude ranging from about 1,100 to 2.300 feet. It is a rugged and hilly upland dissected by the headwaters of numerous streams. Limestone, which dips slightly to the southeast, has provided the resistant erosional surface of the plateau and caps the remnant hills. Between elevations 1,100 and 600 feet, the Balcones fault zone forms an abrupt transition from the hill country in the northwest to the rolling plains in the southeast. The zone is marked by fault escarpments in places, but lacks topographic expression where formations on both sides of the faults are equally resistant to erosion, such as along the tunnel alignment. The fault blocks are composed predominantly of limestone and shale beds which dip gently southeastward. The Gulf Coastal Plain lies below elevation 600 on the downthrown side of the fault zone. It is a rolling prairie underlain largely by beds of clay and poorly consolidated sand. The regional dip is greater in this province, continuing southeastward toward the Gulf of Mexico.
- b. Stratigraphy. The regional stratigraphy consists of Recent to Pliocene aged alluvial deposits underlain by sedimentary formations of the Tertiary to Cretaceous Periods. The alluvial deposits consists of various combinations of gravel, sand, silt, and clay with occasional cobbles and boulders in places. They are predominantly fluviatile floodplain and terrace deposits of which the oldest two have been formally named, the Leona Formation (lower Pleistocene) and the Uvalde Gravel (Pliocene). The underlying Tertiary formations are of the Eocene and Paleocene time epochs. These consist of clay, lignite, sand, and sandstone of the Claiborne, Wilcox, and Midway Groups. Cretaceous formations are contained in the Navarro and Taylor Groups of the Gulf Series and consist mostly of shale, clay shale or claystone, limestone, and sandstone. The Taylor is discussed more fully in succeeding paragraphs as it relates to the project geology.
- c. Structure. The regional structure may be divided into three distinctive areas: The nearly flat and relatively undisturbed beds of the Edwards Plateau; the gently dipping but faulted and folded beds of the Balcones-Luling fault zones; and the southeast dipping monocline of the Gulf Coastal Plain. The rock formations strike east-northeast and dip south-southeast throughout the region. The average formation dip in the Edwards Plateau ranges from 10 to 15 feet per mile, but it increases to 150 feet per mile in the coastal monocline. Between these two areas, the formations dip gently, but are faulted downward about 3,000 feet in a distance of about 22 miles.

Regionally, there are two major fault zones, the Balcones fault zone and the Luling fault zone. The Balcones system contains all of the faults within and north of San Antonio, and is separated by a large graben from the Luling system about 25 miles to the east-southeast. (The Mexia fault zone forms the east side of a similar graben to the north in central Texas.) Both fault zones were apparently part of the same tectonic system which was active during the mid to late Tertiary Period. Normal or gravity faults are predominant in both zones, but the Balcones faults are usually downthrown to the east or southeast and the Luling faults are usually downthrown to the west or northwest. Major faults of both zones trend east-northeastward, roughly parallel to the formation strikes. The almost straight traces of these faults suggest nearly vertical fault planes. Shatter zones are common with numerous small step faults occurring within a narrow area. However, large faults also occur and several are known to have displacements in excess of 100 feet. The Balcones faults have the greatest displacements; a fault northwest of San Antonio, near Helotes, has the largest known throw of about 600 feet, and another fault in south San Antonio has a throw of more than 550 feet.

Although faulting is the more prominent structural feature of the region, the faults generally have decreasing displacements toward the ends of their trace, and in places diminish into folds, especially in the softer strata. A major asymmetrical fold, the Culebra Anticline, plunges southwestward several miles west of the tunnel project. It has a core of Austin Chalk and is flanked by mostly Taylor and Navarro formations. Both flanks of the anticline are terminated by faults of the Balcones system.

3-02. Geology of the Tunnel Alignment.

a. Overburden, Overburden along the tunnel alignment consists of fluviatile low terrace deposits, residual clay, and occasional man-made backfill or construction surfacing. The fluviatile deposits are for the most part clay, clayey gravel, and gravelly clay with lesser amounts of silt and sand. Lower gravel beds are largely composed of calcareous concretions formed around chert or limestone pebbles; these are rounded to subrounded, whitish concretions usually ranging from 1 to 2 inches in diameter, although sometimes as large as 3 inches. A water bearing gravelly clay to clayey gravel is often the basal stratum of the overburden, except where the primary formation is directly overlain by residual clay. The residual clay is tan to buff with gray streaking and mottling, soft, and of medium to high plasticity. It is similar to the underlying weathered clay shale except that it lacks distinct bedding structure and induration. In places, isolated pebbles within the clay suggest possible re-working with the overlying alluvium. Being within a city, the natural overburden is frequently overlain by man-made deposits such as concrete, asphalt, and random soil fill, including minor amounts of construction rubble and other refuse

The overburden blanket, or regolith, along the tunnel alignment varies typically in thickness and character. Overall thickness increases downstream along the tunnel alignment from 1 0 foot at the inlet shaft to 27 feet at the outlet shaft. Individual strata range in thickness from about 1 to 10 feet Although the fluviatile deposits are relatively well sorted from the finer grained deposits near the surface to the coarser gravel deposits at depth, the

gravel beds generally display a good gradation in the engineering sense that various grain sizes are distributed throughout. Cobbles are present in places but never numerous. Clayey gravel often grades into gravelly clay. The clay may be either fluviatile or residual. Both types of clay may range from lean to fat in plasticity and are variably calcareous. The fluviatile clay may

contain gravel, particularly toward the base of the stratum.

b. Primary Formation. The Taylor Formation/Group is the primary and only rock formation encountered throughout the San Pedro Creek Tunnel excavations. Geologic literature often refers to the Taylor as a stratigraphic group containing several formations. Although the formations vary from place to place in composition and name, the Taylor may be generally divided into three stratigraphic units: the Upper Taylor Marl (also called the Marlbrook Marl or Bergstrom Formation), the Pecan Gap Formation, and the Lower Taylor Marl (also called the Sprinkle Formation). Keith Young, May 1965, in referring to these three formations classifies the lithic sequence as: claystone, chalk or marly limestone, and claystone," thereby substituting claystone for the old marl terminology used by Sellards, et al., August 1932. Since "marl" is an old and loosely applied term for unconsolidated or little indurated materials containing 35 to 65% clay and 35 to 65% carbonate (American Geological Institute's Glossary of Geology, 1974), it can apply to the Taylor in composition only. As a geologically consolidated mass of predominantly clay and carbonate minerals, the Taylor is more aptly classified as a calcareous clay shale where fissile, a calcareous claystone where lacking fine lamination, and possibly a marlstone where highly calcareous. Although the Taylor Formation encountered in the tunnel excavations consists of variations and subtle transitions through all three of these similar rock types, we have for simplicity chosen calcareous clay shale as the general project classification of the Taylor rock.

Locally, the Taylor is treated as a formation rather than a group, since only the upper stratigraphic unit is present. However, the formation contains interbedded calcareous or limy layers which may be used as stratigraphic marker beds in electric log correlations. These marker beds have been designated M-1 through M-5, from youngest to oldest. The fifth marker bed, M-5, represents all of the formation below a distinctive 2± foot thick greensand or glauconitic zone. Due to the formation dip to the southeast and the vertical displacement of faulting, the tunnel crosses through four stratigraphic marker beds from the M-1 at the outlet to the M-4 at the inlet, thereby progressing upstream from younger to older beds. This was significant to the tunnel and shaft excavations. Upstream, the formation becomes more limy as it forms a gradational transition toward the underlying Anacacho Limestone and Austin Chalk. X-ray diffraction tests reveal that the stratigraphically lower and older beds tend to be two to three times more limy. The ratio of clay to calcium carbonate is inversely proportional in this material. Thus, the M-1 and M-2 materials are more clayey and lithologically weaker, the M-3 through M-5 materials are typically more limy, better cemented, and more geologically consolidated to give a denser and stronger rock

Although there is only one rock formation encountered by the tunnel construction, its material characteristics are both variable and distinctive.

A rudimentary visual observation can roughly ascertain the variable clay and carbonate (lime) lithology. The darker gray, unctuous, soft to moderately soft material is higher in clay content; the lighter gray, earthy, moderately soft to hard material is higher in calcium carbonate. More exactly, X-ray diffraction indicates that the rock consists of 30 to 45% clay, 15 to 50% carbonates, 10 to 30% quartz, and a trace to 15% of feldspar. The more prevalent of the clay minerals is the expansive montmorillonite with lesser amounts of non-expansive illite and kaolinite, although this is not everywhere the case. Pyrite crystals occur in places, as does calcite and gypsum; the latter two usually form healing minerals along occasional fractures. Marine fossils appear scattered throughout, though more abundant in certain zones. The flat, spirally twisted pelecypod (oyster) "Exogyra" is common. Black carbonaceous specks are found occasionally, and a 0.1-foot thick lense of lignite was encountered at the 99-foot depth during an extensometer installation at Station 158+47. Other than the typical shaly odor, the material often emits a petroleum odor suggesting the possible presence of hydrocarbons and odorless gases. However, the tunnel excavation was continually monitored for explosive hydrocarbon gases and none were detected.

c. Geologic Structure. The Taylor Formation along San Pedro Creek Tunnel consists of about 230 feet of massive and generally undisturbed strata. Boring investigations had nearly 100% core recovery with RQD also approaching 100%. Construction mapping denoted occasional widely scattered fractures and low angle joints, but these are random breaks that hardly disrupt the massive character of the formation. The apparent dips of the joints and fractures is often 1 degree or less with a maximum of 10 degrees; their direction of dipranges from southeast to northeast. The stratigraphic inclination varies along the alignment from 0 to 2 degrees, with the predominant dip to the southeast. Some stress relief fracturing occurred around the excavation openings, and occasional block fallouts were noted during the construction of the outlet shaft transition and the downstream tunnel section. However, the massive character of the formation undoubtedly limited the stress relief effect.

Though the tunnel was excavated in massive rock, the formation is not without structural attributes of the Balcones fault zone. Features of the fault zone are evident in mid-alignment where a high angle fault crosses the apparent flank of a fold dipping to the south-southeast. With respect to the tunnel alignment alone, the fold appears as a faulted monocline. However, with a broader view of the local structure, it could well be that drag flexures were developed on each side of the fault. The strata is essentially horizontal in the upstream third of the tunnel; the mid-tunnel strata dip at 1 to 2 degrees south-southeast; and the beds in the downstream third level out before turning upward to a northerly dip of 7 feet per mile at the outlet. It is this slight reversal in the direction of dip that suggests adjoining flexures resultant from local fault block movements. These flexures may be viewed as upward drag on the downthrown fault block.

Extensive geologic investigations for both tunnel alignments on this project have updated and enhanced the depiction of the stratigraphic and structural geology of central San Antonio. Rather than the one fault which was formerly mapped through the downtown area, this project has revealed four faults

trending east-northeast across the central city between Brackenridge Park to the north and Roosevelt Park to the south. Rather than a fault contact between the Taylor and Navarro Formations (Groups) being near the Paseo del Rio, it is actually just roth of Brackenridge School by about 500 feet. This more complete view of the cal geology lends reason to the development of drag flexures along the tunnel alignment rather than monoclinal folding.

The complete relation of the local structure to the San Pedro Creek Tunnel is made more apparent by projecting vestward the faults which cross the San Antonio River Tunnel. If these faults are projected westward, a down-to-the-south fault crosses the tunnel in mid-alignment, and another down-to-the-north fault passes south of the outlet. This broader view of the tunnel geology reveals a horst and graben structure with the northern alignment in an upthrown horst block and the southern alignment in a downthrown graben block. Therefore, the upward turning of the strata at the outlet could indicate a slight synclinal flexure in the downstream graben with an adjoining anticlinal flexure in the upstream horst. The slight dip in the otherwise horizontal beds would indicate drag relative to the movement of these fault blocks.

The geologic structure displayed along the tunnel alignment, though characteristic of the Balcones fault zone, does little to disrupt the massive character of the rock formation. The folding is but minor warping of essentially horizontal strata. The mid-alignment fault at Station 171+50 has 32 feet of displacement, but has caused little disturbance to the surrounding rock. In fact, the only evidence that the fault was crossed by the TBM was that the muck changed from soft, dark gray, clayey, M-1 and M-2 material to the harder, light gray, limy muck of the M-3 strata. However, though the faulting and folding along the alignment is relatively unimposing, they are both significant in that they place four of the five identified stratigraphic marker beds within the limits of the tunnel excavation.

- d. Formation Weathering. The predominantly tan coloring of weathered Taylor Formation contrasts sharply with the darker, gray unweathered clay shale. The tan coloration is mottled and streaked with gray generally throughout the weathered zone, and rusty stains of oxidized iron occur along some joints and fractures. Though the unweathered formation is massive with few structural breaks, joints and fractures are not uncommon in the weathered zone. It is noteworthy that since there is little water migration through the fractured areas, the top of the weathered zone may be considered the contact between the Taylor aquiclude and the overlying alluvial aquifer. The weathering usually extends through the upper 15 to 20 feet of the formation with an average thickness along the tunnel alignment of 18.5 feet contact with unweathered formation is generally at 30 to 40 feet below ground surface or at an average depth of 34.5 feet. The weathered material is soft, has medium to often high plasticity, is damp in places, and contains scattered fossils. It is distinguishable from the occasional residual clay deposits by slight induration and distinct bedding structure. Due to this induration and bedding structure the material tends to break in blocky chunks when excavated
- e. <u>Ground Water.</u> The Taylor Formation is an impermeable clay-based rock which forms an aquiclude prohibiting the migration of ground water from

both above and below the formation. Ground water in the overlying alluvium is prevented from moving downward, and ground water in the underlying limestones is confined under artesian pressure. The Taylor is a massive tight aquiclude, although there are occasional structural breaks. Where breakage does occur it is usually tight, closed by intrinsic expansive clays, or healed by mineral precipitation. Thus, the impermeable character of the rock is not significantly altered by fractures, joints, or faults. The tunnel excavation was entirely in dry rock with no seepage along structural breaks.

The shaft excavations were also in dry material for the most part. The San Pedro Creek Inlet Shaft was started in unweathered Taylor Formation after the approach channel construction of a previous contract had removed the alluvial overburden and weathered rock; therefore, the inlet shaft was excavated in entirely dry rock. Concrete soldier piers or steel casing was used to seal off any ground water in the alluvial overburden at each of the drilled shafts. The excavation of the San Pedro Creek Outlet Shaft encountered ground water inflow at 200 gpm at the 19-foot depth. Ground water inflow began at the top of a sand stratum that underlay a gravelly clay. The inflow continued at about 200 gpm through 2 to 2.5 feet of the sand and 5 feet of underlying sandy to clayey gravel to the top of the weathered Taylor Formation. This water was removed with sump pumps, and the rest of the excavation was dry.

The main ground-water concern for the tunnel was that the TBM might excavate through an abandoned and unplugged extesian well. The major water source for the region is the Edwards Aquifer, from which the city has a multitude of wells. Occasionally, unknown abandoned wells are found, and there are no assurances that these old well were plugged as required by current regulations. The Edwards lies confined with an arterian pressure beneath the Taylor and other impermeable strata at a depth of about 690 feet, or 550 feet below the tunnel. It has been estimated that an unplugged well from within this aquifer could release as much 23 5000 gpm of water into the tunnel at a pressure of 70 psi. As ir turned out, an abandoned well was indeed intersected by the tunnel excavation, but it proved to be more of a nuisance than a major problem.

The abandoned well was encountered by the TBM at about 2400 hours on May 16, 1989 at Station 178+49, the location of liner ring number 898. The well had apparently been plugged to some extent when abandoned, but it was producing water at a steady 2 gpm, which proved difficult for the contractor to stop. Probing of the inner casing was obstructed at the 24-foot depth by what was probably a remnant of the old plug. The well consisted of a 4-inch diameter inner casing, a 6-inch diameter outer casing, and an 8-inch diameter borehole located 2 feet east of the tunnel center line. The contractor's well plugging events were as follows:

May 17, 1989 -- A professional well driller from T C. Johnson Drilling Company (Weil Digger Licence No. 857) was hired by the contractor to plug the well. First a steel cap with a grouting pipe was welded on top of the inner casing, and then grouting began under the direction of the well driller. After pumping 3 cubic feet of 1.1 grout (water/cement by volume) through the steel cap, grout began flowing between the inner and outer casings. After 6 more cubic feet of 1:1 grout was pumped, grout

leakage developed through cracks in the rock within a 2.5-foot radius of the well. The well driller declared the well plugged after a total of 12 cubic feet of 1:1 grout had been pumped at pressures reaching as high as 150 psi. However, clear water continued to flow out of cracks in the surrounding rock. The contractor allowed the grout to set-up for a couple of hours and then resumed tunneling.

May 18, 1989 -- The well was obviously not plugged since about 2 gpm of water was still flowing into the heading invert from beneath the liner segments. Two holes were drilled through liner ring number 898 to reach the well.

The contractor attempted to plug the well in a fashion similar to the previous attempt. The grouting was stopped after 13.5 cubic feet of 1.1 mix was pumped. It was noted that grout was being forced out between the liner segments.

May 19, 1989 -- Water continued to flow into the heading invert. However, the well was by this time beyond reach or observation since it was beneath the TBM trailing gear.

May to September, 1989 -- The holes through liner ring number 898 were backfilled with pea gravel and left open for observation of the well flow. The flow rate continued through this period at about 2 gpm.

September 28 to October 16, 1989 -- A 5.5-foot long by 3-foot wide by 3-foot deep rectangular hole was excavated around the well The water flow continued at about 2 gpm from the annular space between the outer casing and the borehole

October 16, 1989 -- Once again the contractor attempted to plug the well No water was actually flowing out of the well casing which could only be probed to a depth of 15 feet; this was 9 feet higher than the original probe on May 17, and indicated that previous groutings had sealed off the well casings. Since the water was only flowing out of the outer annular space, the upper well casings were backfilled with 1 1 grout. A grout pipe and a flow pressure relief hose were fixed into the annular space; the grout pipe extended 6 feet below the top of the casing, and the pressure relief hose went about 2 feet into the annular space. An unknown amount of 8:1 to 1.1 grout was pumped into the annular space at 10 psi for about 2.5 hours. Grout leaks persisted in cracks in the surrounding rock, even though saw dust was used as lost circulation material and plugs were driven into leak holes. It was finally decided to let the grout set-up overnight.

October 17, 1989 -- The well's outer annular space continued to leak at 1 to 2 gpm. Grouting was reinitiated in the annular grout pipe, at was shortly stopped in favor of pouring a heavy grout cap over the well. The 5.5-foot by 3-foot hole surrounding the well was filled with heavy grout with grout pipes placed at previous leak locations for future grouting

October 18, 1989 -- The grout cap had set-up overnight, but had water leaks in a few places. Grouting resumed through the pipes installed inthe grout cap. There was a total of 42 cubic feet of 1:1 mix pumped at 93 psi. Some water was noted at joints in the upstream liner segments.

October to December, 1989 -- The grout cap over the well was observed for renewed seepage during this period. The cap became completely dry, and the well was considered plugged.

November 30 and December 1, 1989 -- The upper foot of the grout cap was saw cut and removed. This hole was then backfilled with 6000 psi concrete.

- f. <u>Seismicity</u>. The San Antonio area, as most of southern Texas, is in a Seismic Probability Zone 0. This zero zone extends north-south from Dallas to Brownsville and east-west from Beaumont to Del Rio. No earthquake damage has ever been experienced within this zone, nor should any be anticipated in the future. There are no distant threats from earthquakes beyond this zone. Therefore, the tunnel project has no seismic risks.
- g. Engineering Characteristics of Overburden. The predominant component of the overburden is medium to high plasticity clay though silt, sand, and gravel also occur. The gravel deposits are often clayey to a variable extent ranging from clayey gravel to gravelly clay. Silt and sand layers are also slightly clayey in places. Though the overburden consists of various gradations from fine to coarse materials, it was possible through thorough investigations to develop one set of overburden design parameters for all of the shaft and surface structures. These parameters are as follows:
 - 1 Moist Unit Weight (ym) 125 pcf
 - 2. Saturated Unit Weight (γsat) 130 pcf
 - 3. Shear Strength Assumptions:
 - a. Cohesion (c') 0.1 tsf
 - b. Angle of Inner Friction (σ') 20°
 - 4. Allowable Bearing Capacity (qall) 2.0 tsf
 - 5. Earth Pressure Coefficients:
 - a. Ka (active) = 0.5
 - b. Ko (at rest) = 0.7
 - c. Kp (passive) 2.0
 - 6. Modulus of Subgrade Reaction

or Spring Constant (Ks) - 75 pci

h. Engineering Characteristics of Primary Formation. The characteristic of the primary formation which caused the greatest design concern was its capability of exerting relatively large swell pressures on tunnel and shaft linings due to its montmorillinite content. Although the swelling pressure is very low in some of the material and is usually less than 5 tsf, it is known to be as high as 15 tsf in places. Therefore, geotechnical consultants were engaged as advisors during the tunnel and shaft design. The swell pressure characteristics and the recommendations of the consultants are discussed in Part IV, Special Design Considerations, Paragraph 4-02

Other engineering characteristics were determined for selected undisturbed samples along the tunnel alignment. In Atterberg tests, the average liquid limit was 50 with a high of 75 and a low of 34; the average plastic limit was 17 with a high of 19 and a low of 14; the plasticity index averaged 33 with a high of 56 and a low of 20. The moisture content ranged from 6% to 15.8% with an average of 10.5%. Specific gravity was about 2.70. Dry density ranged from 116 pcf to 140 pcf with an average of 129 pcf. Unconfined compressive strengths near the tunnel depth varied from 25.6 tsf to 132.8 tsf, averaging 71.4 tsf. The soil modulus near tunnel depth ranged from 2.2 X 10⁴ psi to 19.8 X 10⁴ psi, with an average of 9.1 X 10⁴ psi.

A set of design parameters were developed for both the weathered and unweathered primary formation, noting characteristic changes with depth. These parameters are as follows:

Weathered Shale (undisturbed)

- 1. Moist Unit Weight $(\gamma m) = 125 \text{ pcf}$
- 2. Saturated Unit Weight (ysat) = 130 pcf
- 3. Shear Strength Assumptions:
 - a. Cohesion (c') = 0.1 tsf
 - b. Angle of Inner Friction $(o') = 25^{\circ}$
- 4. Allowable Bearing Capacity (qall) = 3.0 tsf
- 5. Earth Pressure Coefficients:
 - a. Ka (active) = 0.4
 - b. Ko (at rest) = 0.9
 - c. Kp (passive) = 2.5
- 6. Modulus of Subgrade Reaction or Spring Constant (Ks) = 250 pci

Unweathered Shale (undisturbed)

- 1. Moist Unit Weight $(\gamma m) = 135$ pcf
- 2. Saturated Unit Weight (γsat) = 140 pcf
- 3. Shear Strength Assumptions:
 - a. Cohesion (c') = 0.1 tsf to 0.5 tsf @ tunnel depth
 - b. Angle of Inner Friction (o') = 35° to 45° @ tunnel depth
- 4. Allowable Bearing Capacity (qall) = 6.0 tsf

(Note: The allowable bearing capacity for the unweathered shale actually exceeds 6.0 tsf at tunnel depth, but with no effect on structural design.)

PART IV

SPECIAL DESIGN CONSIDERATIONS

4-01. Construction Method. The tunnel concept for flood diversion beneath the city was adopted rather than surface channel modifications to avoid construction impacts to the downtown area. Significant costs and liabilities would ensue from surface construction along the drainage channel due to limited access, potential damage to structures, bridge replacements, traffic congestion, business restrictions, and other city related problems. However, though convenient from a construction standpoint, the tunnel method along San Pedro Creek was necessarily incorporated with the flood control tunnel planned for the downtown reaches of the San Antonio River. Because of the high cost of a tunnel boring machine (TBM) and initial mobilization expenses, the cost per foot of tunnel is substantially decreased as the length of tunneling increases. The 5,985-foot long San Pedro Creek Tunnel would hardly have been cost effective without the additional 16,200-foot length of the San Antonio River Tunnel. Therefore, without the added length of the San Antonio River Tunnel, the San Pedro Creek project would have been restricted to surface channel improvements, or less expedient but lower cost conventional methods of tunneling.

A fully shielded, mechanical tunnel excavating machine was specified for the contract which included both the San Pedro Creek Tunnel and the San Antonio River Tunnel. The contractor was given the choice of using a full-face tunnel boring machine (which was chosen), a boom header machine, or a roadheader machine; the latter two would have been allowed only if fully shielded and equipped with an excavation guide ring.

The contractor was also given the option of following the excavating machine with cast-in-place concrete liner or precast concrete segmental liner, provided that the installation of either left no ground unsupported behind the shield. The precast segmental liner was the selected method, providing both initial and final support. The contractor was also given the flexibility to design the liner erection and support method, although the contract plans presented a method using longitudinal needle beams and steel ribs. The method of liner erection was specified to provide "positive structural support" to prevent deviation from circularity of the segmental rings and to prevent settlement of the rings into the invert void as the segments left the back of the tail shield. The contractor's designed method was to set invert segments on a bed of pea gravel, use interlocking dowels between segment rings, support segments at springline with wood blocking, and finally blow pea gravel around the entire ring to provide positive structural support. The lower portion of the tail shield behind the grippers was removed to facilitate this operation.

The specified shaft excavations also allowed the contractor flexibility in selecting a preferred method of construction. The inlet, outlet, and maintenance shafts could be excavated by mechanical ripping, controlled blasting, or a combination of these techniques. Actually, the maintenance shaft was excavated by rotary drilling, and no blasting was used on any

portion of the San Pedro Creek project. The small diameter shafts for ventilation and hydraulic instrumentation were specified for drilling with the option of proceeding downward from the surface or upward from the tunnel (raise drilling). These were drilled downward from the ground surface.

4-02. Swell Pressures. The swelling potential of the primary formation was a major design consideration, especially in the determination of strength requirements for the tunnel and shaft liners. Laboratory testing during design investigations indicated that the material was capable of exerting expansion pressures considerably larger than the overburden pressure. Swell pressures of as much as 12.8 tsf were recorded with a maximum overburden pressure of 8.8 tsf at a depth of 135.3 feet. However, it was questionable as to whether the tunnel and shaft liners would actually have to withstand field pressures as great as those indicated by the laboratory constrained testing. In support of this questioning was previous swell testing by Dr. Tor Brekke on Taylor material from the Austin Crosstown Wastewater Interceptor. Dr. Brekke's tests had shown that permitting the material to experience a volume increase of 2 % reduced the swelling pressures by roughly 50%. On the other hand, the montmorillonite content of the Taylor in Austin varied somewhat from that of the Taylor in San Antonio tunnels. Therefore Dr. Ralph Peck was engaged by the government as a consultant in resolving these questions and other geotechnical issues throughout the tunnels project.

At the recommendation of Dr. Peck, Dr. G. Mesri of the University of Illinois was enlisted to do further testing and evaluation of the Taylor swell properties from samples taken along the tunnel alignments. Based on the previous design tests, field observations, and Dr. Me_ri's tests, both consultants recommended that the tunnel and shaft liners should be designed to withstand swell pressures of 5 tsf.

The reasoning of the consultants was that the potentially high expansion pressures indicated by laboratory testing would be largely dissipated as the swelling material expanded into space provided by stress relief fissures that inevitably develop around underground excavations. In Dr. Peck's words, "...the stress release associated with excavating the tunnel of 20-feet (26.9 feet) diameter would undoubtedly be sufficient to cause the opening of fissures around the tunnel to an extent that the ultimate swelling pressures would be reduced to the design value (5 tsf). These fissures would be developed by the time the tailpiece of the shield would expose the shale." Likewise, Dr. Mesri concluded that laboratory pressures would not develop in reality against the tunnel liner because the magnitude of shale rebound after excavation would open fissures around the tunnel periphery. He also expected swell pressure dissipation due to expansion into the tunnel's annular space about the lining, due to flexibility of the lining itself, and due to partial swelling of the material before the lining could be installed Dr. Mesri's tests produced swelling pressures ranging from 0.2 tsf to as high as 15 tsf, although more than 2/3 of the results were less than 5 tsf. (This broad range is indicative of the variable montmorillonite content throughout the formation.) However, similar to Dr. Brekke's findings, he found that to allow additional swelling in a laboratory specimen above the initial void ratio, corresponding to 0.35% axial strain, reduced the swelling pressure from 8 tsf

to 4.5 tsf. Therefore, it was concluded that the inherent field conditions in tunneling would reduce the actual swell pressures on the lining.

Although Dr. Mesri estimates from calculations of the time-rate of swelling that the total design pressure will require decades to develop, experience within the San Antonio area suggests that a substantial amount of the swelling can be expected within 5 years. Based on local experience, it is anticipated that most of the 5 tsf may be realized upon the tunnel and shaft liners within 5 to 10 years after construction. Expansion is usually negligible beyond 12 to 15 years after the moisture environment is changed.

4-03. Heave Potential. Another design consideration was vertical uplift or heave due to differential expansion of the material surrounding the shafts. Since the percentage of expansive montmorillonite varies within the primary formation, the amount of swelling can vary throughout the shafts. Also, moisture variations can affect the rate of swelling from place to place. Particularly, the upper weathered formation is likely to swell more rapidly than the unweathered material at lower depths. Therefore, to deal with possible vertical displacements or tensile forces developed by these conditions, the designers recommended that the shafts be constructed with expansion joints, tensile steel, and/or a bond breaker between the permanent and temporary liners.

A shaft bond breaker was specified for the Phase II tunnel contract. (An expansion joint was included in the surface structure design to be constructed under a later Phase III contract.) The specified bond breaker was a geotextile material which was to be installed over the initial support. However, a contract modification provided a substitute for the geotextile which consisted of an asphalt fiber board, Sealtight Dummy Joint, produced by W.R. Meadows, Inc of Fort Worth, Texas.

EXCAVATION AND SUPPORT PROCEDURES

5-01. <u>General</u>. The contract required that the San Pedro Creek Tunnel and Shafts be completed first, although the San Antonio River Tunnel and Shafts could be started concurrently. There was no differentiation for payment in types of material excavated such as rock or common excavation; payment for shaft excavation was lump sum for each shaft, and payment for tunnel excavation and lining was by the linear foot. San Pedro Creek Tunnel and Shafts involved payment for 5,842.86 Ifnear feet of tunnel excavation, a like amount of precast segmental liner, and lump sum for each of 7 shafts.

Most of the tunnel and shaft excavations closely followed the lines and grades indicated in the plans and specifications. The specified tolerances for the tunnel excavation allowed an alignment departure of ±12 inches, a grade departure of ±3 inches, and a rate of return to alignment or grade not greater than 3 inches per 100 feet. The contract required that the vertical and horizontal tunnel alignment be controlled by laser beam instrument. Although numerous line and grade adjustments were required in controlling the TBM, particularly in negotiating the curve sections, the overall results were quite accurate; the tunnel hole-through at the inlet shaft was little more than an inch northeast of the alignment. No variations were allowed in the thickness of the tunnel lining. The precast segmental liner was allowed a variation of 0.5% from the inside dimension, an out of roundness of $\pm 3/4$ -inch in diameter, and abrupt irregularities at segment joints not in excess of 1/4-inch. The shaft excavations were allowed 0.5% of the depth in out-of-plumbness or 10% of the finished inside diameter for circular shafts, whichever would be less. Variation from the excavated diameter of circular shafts could not exceed 0 to plus 6 inches. Shaft linings were allowed a variation in thickness of minus 2.5% or 1/4 inch, whichever was greater. The inside dimensions of shaft linings were given a tolerance of 0.5%.

In addition to establishing the lines, grades, and dimensions for the tunnel and shafts, the plans and specifications provided a guideline for implementing the construction. However, the Contractor had the option of submitting for approval his own design proposals for excavation and support. When approved by the Contracting Officer, the Contractor's design and procedures became the de facto specifications in their applicable areas of construction. Each area of construction and the procedures used will be described in the following paragraphs.

5-02. Excavation Equipment,

a. <u>Shaft Excavation Equipment</u>. Two types of equipment were used for the shaft excavations. Hechanical ripping equipment was used in the inlet and outlet shafts, drilling equipment was used in the maintenance, vent, and hydraulic instrumentation shafts. In the inlet and outlet shafts the downward vertical excavation was accomplished by backhoe, but a roadheader was used for outward extensions of the shaft walls and for undercutting the horizontal

transition toward the tunnel. The harder limy-layers in the inlet shaft were broken through by using a hydraulic ram attached to a backhoe. The other five shafts were rotary drilled with a 45 ton Northwest 5045 crane rig. The following is a list of the actual equipment used during the shaft excavations:

Excavation and Mucking

JD 490 Backhoe	TB-45 Excavator	
Cat 235 Backhoe	Mitsui Road Header	
Cat 205 Backhoe	Cat Loaders 988, 966,	
Yamashi Backhoe	950, 931, 920	
Mitsubishi Backhoe	JD 455 Loader	
Yutani Backhoe	Case Bobcat Loader	
Takeuchi TB-45	Cat IT-28	
(with hydraulic ram)	630 Rocker (mine mucker)	

Cranes

Maniferen	4600
Manitowoc	4600
Northwest	5045
Manitowoc	3900
American	165 tor
Linkbelt	100 tor
P&H	90 ton
Grove	35 ton
Linkbelt	20 ton
Gallion	18 ton
Clark	15 ton
Drott Deck	Crane

b. <u>Tunnel Boring Machine (TBM)</u> The entire tunnel was excavated with a modified Robbins Model 243-217 tunnel boring machine. The machine had been originally designed for hard rock tunneling, and had been previously used to excavate the Kercknoff 2 Tunnel in the Sierra Nevadas near Fresno, California. Ohbayashi engaged Boretec, Inc. of Solon, Ohio to renovate and modify the machine for the soft rock tunneling in San Antonio.

The TBM was converted from an open-faced hard rock machine to a fully closed soft rock machine with articulating shield. A new main beam was installed to shorten the machine and to help moderate the machine weight. The front support shoe was tripled in length to better distribute the machine weight which increased from 380 tons in the original machine to 550 tons with the Boretec modifications. The cutterhead was enlarged from a diameter of 24 feet-1 inch to 26 feet-11 inches, this gave a tunnel annular space behind the liner of 3.5 inches. The main bearing was replaced providing an increase in cutterhead thrust capacity from 1,166 tons to 1,547 tons. The side-gripper shoes were enlarged to 56 inches by 138 inches for a better dispersing of forces exerted on the tunnel sides. As an auxiliary propulsion system, 12 thrust cylinders were added with thruster shoes for pushing off of the liner segments; these thrusters could also be used to hold the precast segments during the liner erection. A ring-type segmental liner erector was added within the back of the tail shield. The back 57 inches of the lower

 120° section of the tail shield was cut away to allow the placement of the invert segment on a bed of pea gravel.

Although a complete description of the TBM would be too voluminous for this report, there are several additional features which should be noted. When fully operational in the San Pedro Creek Tunnel, the TBM and it's trailing gear was 274 feet long; the length from cutterhead to end of tail shield was 38 feet. The cutterhead contained 57 disc cutters of 15.5-inch diameter. The outermost 7 discs were the gauge cutters which determined the final sizing of the tunnel bore. The outer perimeter of the cutterhead contained 12 bucket scoops which collected the muck and dropped it into the conveyor system within the cutterhead support. The drive torque for the cutterhead assembly was provided by 10 single-speed, 3-phase, AC electric motors producing 200 HP (149 KW)each. These motors rotated the cutterhead clockwise, in an upstream view, at 5.75 RPM. The four main propulsion cylinders, hydraulic jacks, generated horizontal thrusts at 7.5 degrees outward from the tunnel's longitudinal axis resulting in a forward machine thrust and a side thrust on the gripper pads. This system could generate a total thrust force of 2.64 X 106 lbs.

Two methods of TBM propulsion were provided since it was anticipated that some of the ground would be too soft, or weak, to withstand the thrust and shear forces exerted through the side grippers. In the stronger, stable ground the four main propulsion cylinders could propel the machine by pushing the side grippers against the tunnel wall. This method does not interfere with preparations for segmental liner erection in the invert area at the back of the tail shield. In ground too weak to withstand propulsion through the side grippers, the machine could be propelled by 12 auxiliary jacks shoving against the segmental liner. However, the shove jacks in this method obstruct the working area at the back of the tail shield.

5-03. <u>Precast Tunnel Liner.</u> The tunnel liner, which also provided the initial support, consisted of precast concrete segments installed within the protective covering of the TBM tail shield. There were 6 segments in each complete ring of liner, forming an inside diameter of 24 feet-4 inches. Each segment was 4 feet wide by 1 foot thick, weighed 8800 pounds, and extended 13.78 feet along a 60 degree arc on the outside of the liner. The bottom 3 segments were ide ical in shape. The top 3 segments were skewed 7 degrees off longitudinal act he two upper joints to accommodate a trapezoidal "key" segment in the crown. The segments were cast of 6000 psi reinforced concrete, and contained two 2-inch diameter grout holes positioned 4.0-feet lengthwise to each side of the center These grout holes were also used for erector handling and for injecting pea gravel into the annular space.

Two types of joints were formed by the segment rings. Circumferential joints divided the rings at 4-foot intervals along the tunnel alignment. Longitudinal or radial joints were formed where the segments joined at each 60° arc of the ring. These longitudinal joints were a tongue and groove type designed by the contractor rather than the specified knuckle type. All of the joints contained a 3/4-inch deep by 1/4-inch wide groove on the inside liner surface for sealant application. The sealant used by the contractor was Sikaflex-lA rather than the specified Hornseal.

The segment rings were aligned and locked together at the circumferential joints with "fast-lock dowels" patented by the segment manufacturer, Sehulster Company, Inc.. These dowels were intended to prevent joint spreading and to make the segment rings free-standing. Each circumferential joint contained 18 equally spaced dowels, 3 per segment.

The segmental liner was installed with a circular erector arm at the back of the tail shield. The erector picked each segment up at the invert and rotated it to its proper position within the ring. As the TBM excavated forward. exposing 4 feet of invert rock in the cut away section of the tail shield, a 3-inch thick piece of flexible styrofoam was set on the invert about 3 feet-9 inches in front of the previous ring. Normally, a bed of pea gravel was placed and graded behind the styrofoam barrier in preparation for the invert segment. At times, however, when the tunnel bore was too high, the invert rock was excavated to grade-cut with pneumatic spades, and no pea gravel was required. The invert segment would then be placed with the erector and pushed onto the dowels of the previous ring by the auxiliary propel jacks. This was followed by the placement of each of the two lower rib segments , which were backed by the styrofoam barrier and supported by wood blocking at springline. The upper two rib segments would then by placed, followed by the installation of the key segment in the crown. No styrofoam barrier was placed above springline. After the full ring was erected, pea gravel was blown over and around the back of the segments or through the grout holes. The pea gravel was intended to provide the primary positive structural support. However, final stabilization of the liner was provided with backpack grouting after the trailing gear had cleared the segments. Complete grouting of the full annular space was generally achieved at about 200 to 250 feet behind the trailing gear (500 feet from heading), although this fluctuated considerably.

5-04. Foundation Preparation. The contract requirements for foundation preparation were specified for the most part under technical provisions for placing cast-in-place structural concrete. Of course this did not apply in the tunnel because precast concrete segments were installed immediately behind the TBM tail shield, rather than lining the tunnel with cast-in-place concrete. Neither did it specifically apply to the large diameter shafts (outlet, inlet, and maintenance shafts) because the rock was initially supported with shotcrete long before the structural concrete was placed. Nevertheless, the specifications state that, "Shale or clay shale surfaces upon which concrete is to be placed shall be clean, free from oil, standing or running water, ice, mud, drummy rock, coatings, debris, and loose semi-detached or unsound fragments."

Actually, these conditions were generally met before shotcrete applications, largely due to practical workmanship. The excavation and support procedures in the large diameter shafts consisted of shotcrete applications after every 5 to 8 feet of vertical excavation. This procedure prevented long term exposure and corresponding deterioration of the rock. The rock was massive and excavated very smoothly, especially with the roadheader, therefore, there were normally no loose blocks or drummy areas in the foundation. Occasional loose fragments were scaled away from the shaft walls before shotcreting. Since it was imperative to provide full contact between the initial support and the

surrounding rock, all over-excavations were fully backfilled with shotcrete as required by the specifications.

The specifications also required that the excavated surfaces of the shafts be protected immediately upon exposure with a polyvinyl acetate emulsion resin containing at least 60(±1)% total solids by weight. Some effort was necessary in enforcing this requirement as well as assuring beneficial applications. Aerospray 70 (or an approved equal product) produced by American Cyanamid Company was specified, but no water dilution mixture was stipulated. The only application requirements were given under the specification section on preparation for cast-in-place concrete placements. An "expert" with the supplier reportedly recommended a sealer to water ratio of 1:20 with an application rate of 1/4 gallon per square yard. However, this mixture appeared too watery with inadequate results, and the contractor eventually increased the ration to 1:10. Where the material was more limy and less susceptible to air slaking the contractor was allowed to omit the resin application if shotcreting was conducted expeditiously.

5-05. <u>Outlet Shaft Excavation</u>. The outlet shaft was excavated and supported according to the contractor's approved design submittals. The 150-foot deep shaft is boot-shaped consisting of an initial vertical section, an intermediate upstream undercut, and finally a tapering 60-foot lateral transition to the tunnel. The entire shaft was excavated by backhoe and roadheader with no blasting required, although the specifications provided for that option. The backhoe was generally used in the vertical excavations whereas the roadheader was used for undercutting or lateral excavations. The initial support was designed by the contractor for a specified rock pressure of 5 ktps.

The excavation began with the construction of a collar in the upper 12 feet of the shaft. This upper portion was excavated to a 51-foot surface diameter tapering downward to a 48-foot diameter at the 12-foot depth. As this initial hole was dug, the collar structure consisting of four W12 X 58 steel rings and wood lagging was preassembled on the ground surface. The rings were held 3 feet apart by the vertically placed lagging to form a 12-foot high, open-ended wooden barrel with a 43-foot inside diameter. The collar structure was then placed within the completed hole, and the annular space was backfilled with concrete.

The next 57 feet of shaft, from the bottom of the collar at elevation 569.56, was excavated to a diameter of 42 feet 4 inches, and was variously supported with steel rings, wood lagging, shotcrete, and wire mesh. W8 X 48 steel rings were installed on 4-foot centers through the overburden and weathered clay shale to the 40-foot depth Generally, a 5-inch thickness of 3500 psi shotcrete was applied between the steel rings except where a groundwater inflow of 200 gpm was encountered in the alluvial aquifer lying between elevations 620 and 612 Wood lagging was installed between the rings located at elevations 619, 615, 611, and 607; grouting was then conducted behind the lagging to seal off the ground water Below the 40-foot depth no steel rings were used, but the shotcrete increased to a thickness of 8-inches with the reinforcement of two layers of 6 X 6 - W6 X W6 welded wire fabric.

A single W8x48 steel ring was installed at elevation 569.56 just before the shaft excavation began to widen and undercut upstream toward the tunnel portal. As the shaft was progressively widened with depth, its cross section in plan view became increasingly egg shaped. In plan view, the downstream half of the shaft remained circular whereas the upstream portion elongated to form an elliptical curve. In longitudinal cross sections, this intermediate undercutting between the vertical shaft and the horizontal transition had the shape of an elbow flexure, and thus was called the shaft elbow. The elbow curvature continued to the crown elevation of the transition, 532.59, or a depth of 107 feet. Below this depth the shaft was excavated vertically to invert with a continuous longitudinal diameter of 70 feet 11 inches and a continuous transverse diameter of 49 feet 6 inches.

The initial support below elevation 569.56 consisted of a 12-inch thickness of 3500 psi shotcrete reinforced with two layers of 4 X 4 - W4.7 X W4.7 welded wire fabric. Also, 18 to 21-foot long rock anchors were installed generally on 4 to 5-foot centers and predominantly in the upstream elongated portion of the shaft. These anchors were 1.25-inch diameter, No. 10 Dywidag threadbars cement grouted into 5-inch diameter holes. They were the primary support where the radius of curvature exceeded 30 feet, or where the excavation had no curvature.

The lateral transition excavation extended 60 feet upstream from the vertical shaft at Station 141+98.14 to the tunnel portal at Station 142+58.14. The transition crown and invert elevations at Station 141+98.14 were 532.59 and 490.34, respectively. The transition crown and invert elevations at Station 142+58.14 were 522.05 and 490.46, respectively. Thus, the diameter of the transition tapered from approximately 42 feet at the shaft to about 32 feet at the tunnel portal.

The transit'on was excavated in four benches in conjunction with the lower 42 feet of vertical shaft excavation. Each of the approximately 10 to 8-foot high benches were cut when the vertical shaft had been excavated to the bottom of that respective level. After the full 60-foot length of the transition was excavated and supported for a particular bench, the vertical shaft was taken down another 10 feet to the bottom of the next bench, and so on to invert.

The transition excavation was supported with W10 X 49 steel ribs and 12 inches of 3500 psi shotcrete. Wood blocking was used only in places to insure that the ribs were making full contact with the surrounding ground; all other gaps between the ribs and the ground were filled with shotcrete. There were 16 of the steel ribs labelled A through P with Rib A set in the first 1 5 feet of the transition, Ribs B and C set on 3-foot centers, and the remaining ribs set on 4-foot centers.

The shaft collar was set between elevation 638.8 and 626 8 on January 29, 1988. Thereafter, the excavation proceeded in 3 to 8-foot vertical tiers, and reached the bottom elevation of 488.0 on August 8, 1988. The lateral transition excavation was completed 4 days later on August 12, 1988.

5-06 <u>Inlet Shaft Excavation</u>. The inlet shaft excavation followed lines and grades similar to those presented in the contract drawings except that

adjustments were made to allow for a 4-inch enlargement of the final inside diameter. The inside diameter of both the inlet shaft and the tunnel were changed from 24 feet to 24 feet 4 inches. The shaft was excavated by backhoe in 4 to 9-foot deep tiers. A hydraulic ram was attached to the backhoe when necessary to break through layers of harder limy clay shale. The primary support was according to the contractor's approved design which allowed for a specified rock pressure of 5 kips.

The first work required at the inlet shaft site was to dewater the approach channel. The inlet shaft was the only large shaft constructed within the actual channel of San Pedro Creek. A concrete approach channel had already been constructed under a previous contract, and was filled with water to a depth of about 15 feet. To dewater the work site, the water was pumped out of the approach channel; a back-flow dike was built downstream from the site; an upstream dam was constructed of steel beams placed across the piers of the Quincy Street Bridge; and water was bypassed from the Quincy Street dam to the back-flow dike through a 30-inch diameter steel culvert. Also, a sump and large trash pump were installed within the dewatered approach channel to remove water from leaks or overflows.

The previous approach channel construction had removed the overburden and weathered clay shale at the site. Therefore, the contractor had only to remove the rip rap, channel concrete, and a few inches of material to begin the shaft excavation in massive unweathered clay shale.

The upper portion of the shaft excavation was in the shape of an equilateral rhombus, but was nearly square with a width of 31 feet 2 inches. It extended to a depth of 21 feet from elevation 623 to 602. The initial support was 3 inches of shotcrete designed mostly to prevent desiccation and air-slaking of the clay shale. Additional support was provided by 24 rock anchors installed on 5-foot centers at each of two elevations, 617 and 612. These anchors were 8-feet long, 3/4-inch diameter, and fully resin grouted.

After the upper shaft was excavated to elevation 602, a 24-foot high by 52-foot diameter circular water protection cell was constructed around the work area. The cell was erected to prevent flooding until a temporary concrete surface structure could be built over the shaft. The cell resembled a large, open-ended, wooden barrel similar to the structure constructed for the collar at the outlet shaft. However, this barrel structure was set on the ground surface around the excavation. The cell was constructed of 5 steel rings held apart by 6-foot long wooden lagging placed lengthwise between the rings. The steel rings were W12 X 58, and the wooden lagging was actually 6-inch by 8-inch railroad ties. The outside of the cell was overlain with a layer of visqueen to help make it water tight. The base was anchored into the ground by No 11 rebar dowels driven through 24 selectively spaced holes in the bottom steel ring. The base was then shotcreted on both sides. The cell leaked during approach channel flooding, but not profusely

The upper 21 feet of excavation provided the foundation for the temporary concrete surface structure. The structure began within the shaft at elevation 603.43, and had the same rhombus shape as the excavation. The entire structure was constructed of reinforced concrete, which included a 3 0-foot

wide by 3.5-foot deep collar at the ground surface. The structure extended 33 feet above the creek channel to elevation 656, slightly above the 100-year flood level of 655.2.

The shaft excavation gradually changed from the rhombus shape at elevation 603.43 to circular at elevation 578.0. Thus, the radius of curvature at the corners changed from zero at elevation 603.43 to 12 feet 2 inches at elevation 578.0. This portion of the excavation was also supported with shotcrete and rock anchors. The shotcrete design was a 5-inch thickness of 3500 psi shotcrete reinforced with 6 X 6-W2.9 X W2.9 welded wire fabric. The rock anchors were designed as additional support for the straight or uncurved sides of the shaft; this gave progressively fewer rock anchors with depth as the shaft became more circular. The anchors were installed across straight wall sections on 5-foot centers and in 5-foot tiers with depth. The number of rock anchors installed at each respective elevation were 20 at 601, 16 at 596, 12 at 591, 8 at 586, and 4 at 581. These anchors were 18-foot long, 1.25-inch diameter, No. 10 Dywidag threadbars cement grouted into 5-inch diameter holes.

The excavation between elevation 578.0 and the shaft elbow at elevation 553.9 was a vertical circular section supported by 5 inches of 3500 psi shotcrete reinforced with one layer of 6 X 6-W2.9 X W2.9 welded wire fabric. No rock anchors were required in this section.

Below elevation 553.9 the elbow curvature of the shaft began to undercut toward the tunnel portal. Unlike the outlet shaft, this shaft was the same diameter as the tunnel, and required no transitional tapering between the elbow section and the tunnel portal. The excavation below elevation 553.9 was initially planned to stop at elevation 517, about a foot below tunnel springline, and thereby allow the TBM to excavate the remainder to invert when it holed-through into the shaft. However, the shaft excavation continued to elevation 508, which left only 3.6 feet for the TBM to excavate to invert elevation 504.4.

The elbow excavation was supported with shotcrete and rock anchors. The shotcrete was 8 inches thick and reinforced with one layer of 4 X 4-W4.7 X W4.7 welded wire fabric. In the downstream section of the shaft, where the radius of curvature exceeded 15 feet, rock anchors were used for added support. These were 15-foot long, 1.25-inch diameter, No. 10 Dywidag threadbars cement grouted into 5-inch diameter holes. The anchors were generally spaced on 4 to 5-foot centers and perpendicular to the shotcreted wall. However, in the crown, or "brow," of the elbow curvature they were inclined upward at 37°.

Excavation of the San Pedro Creek Inlet Shaft began at elevation 623 in the creek channel on October 10, 1988. The rhombus shaped upper portion of the excavation was completed to elevation 602 on October 18, 1988. The temporary concrete surface structure was then constructed after which the shaft excavation resumed on January 6, 1989. The next section, which was a transition from rhombus to circular shape, was completed at elevation 578 on February 2, 1989 The shaft excavation was finished at elevation 508, 3.6 feet above the invert, on June 19, 1989. The TBM hole-through was on July 13, 1989.

5-07. <u>Maintenance Shaft Excavation</u>. The maintenance shaft excavation was performed according to the contractor's approved submittal, which generally provided the specified shaft dimensions. The excavation was accomplished primarily by two drilling subcontractors between May 9 and August 11, 1988.

Cato Electric and Drilling began the work by drilling a ring of 27 concrete soldier piers around the shaft circumference. These 36-inch diameter piers were intended to provide initial support through the alluvial overburden into the underlying weathered, but impervious, clay shale. At Ohbayashi's field discretion, however, the piers were extended through the weathered clay shale into the underlying unweathered formation at depths of 36 to 42 feet. The procedure was to auger every other pier, and backfill it with 3000 psi concrete. The intermediate piers were then augered with a minimum of 1-inch overlap on the adjacent piers, and likewise backfilled with 3000 psi concrete. This overlapping established an 8-inch bearing surface from pier to pier, and provided a ground water barrier through the alluvium.

The 21.5-foot wide interior of the soldier pier ring was then excavated by Ohbayashi with a backhoe. To prevent any possible inward movement of the piers, W8 X 35 steel rings were installed at ground surface, at about the 15-foot depth, and at about the 30-foot depth. The backhoe excavation continued below the piers to the 50-foot depth, enlarging the diameter to 22 feet. Below the piers, the excavation was supported with a 6-inch nominal thickness of shotcrete.

Beck Foundation Company drilled the remainder of the shaft with a Northwest 5045 crane-type rotary drilling rig. A 3-foot diameter pilot boring was first drilled to the 122-foot total depth. Then progressively larger bores of 4 feet, 6 feet, and 8 feet were drilled to various depths. After reaching an 8-foot diameter the shaft was enlarged by progressively reaming to diameters of 11 feet, 16 feet, 19 feet, and finally to 22 feet 4 inches. The 6 nominal inches of shotcrete support was generally applied when a 7-foot deep tier had been reamed to the final diameter. The pilot bore served as a catchment for the drill cuttings, and was cleaned out periodically with an auger.

The shaft was excavated 122.0 feet from the ground surface elevation of 642.5 to a bottom elevation of 520.5. This placed the shaft 7.5 feet below the crown elevation of the unexcavated tunnel. The shaft was then backfilled with sand to elevation 530. This allowed the final concrete liner to be placed upward from that elevation to an inside diameter of 18.0 feet.

The intersection of the maintenance shaft with the tunnel was excavated to tunnel springline for approximately 16 feet to each side of the shaft centerline. The excavation was done by roadheader, backhoe, and pneumatic spaders in advance of the TBM tunneling, and extended from Station 181+58 to Station 181+90. It was supported with W8 X 48 steel ribs set on 4-foot centers, shotcrete as needed, and wooden lagging. The lower half of the tunnel was supported by the precast concrete liner as the TBM completed the excavation below springline. Finally, the upper half of the tunnel and the shaft intersection were formed and cast with 4000 psi reinforced concrete.

5-08. Vent Shaft Excavations. The vent shafts were excavated and supported according to the contractor's approved submittal. Two 6-foot diameter drilled vent shafts were specified for San Pedro Creek Tunnel, and were to be lined with a 4-foot inside diameter precast concrete pipe. However, to connect the tongue and groove pipe joints with 0-ring gaskets would have been somewhat difficult, as would the inspection in these deep, narrow shafts. Therefore, the Government approved the contractor's proposal to install a 4-foot inside diameter, 3/8-inch thick, steel casing from the ground surface. The general shaft dimensions were not changed.

In May 1988, Beck Foundation Company augered both vent shafts using a Northwest 5045 crane-type rotary drill rig. The first vent shaft was located just north of Durango Street at tunnel Station 158+14.13, and was drilled to the 121.0-foot depth. The shaft was then backfilled with drill cuttings to the 117.7-foot depth, to which depth the permanent steel casing was seated. The second vent shaft was located near the intersection of Camaron and Salinas Streets at tunnel Station 185+73.90, and was drilled to the 117.0-foot depth This shaft was also backfilled with drill cuttings to provide a seat for the permanent steel casing at the 114.0-foot depth.

The general construction procedure for each shaft was to auger an oversized bore through the alluvial overburden and set a temporary surface casing into the impermeable clay shale. The remainder of the shaft was then augered to a minimal 6-foot diameter, and backfilled with drill cuttings to the permanent casing depth, about 5 inches above the projected tunnel bore. The 4.0-foot inside diameter steel casing was installed with the 1.0-foot wide annular space backfilled with 3000 psi concrete. The temporary casing was removed as the concrete backfill approached the ground surface.

No further excavation was required for the intersection of the vent shaft and the tunnel, other than minor spading for a concrete ring beam at the junction. The TBM excavated through the bottom of the shafts removing the backfill cuttings through the mucking system. As the precast segmental liner was erected through the shaft area, the crown key segments were omitted and replaced by W6 X 20 steel sets and wood lagging. At the Durango Street shaft five key segments were omitted between Stations 158+10 and 158+30. However, at the Salinas Street shaft only one key segment at Station 185+74 was omitted. The intersections were later formed and cast with 4000 ps1 reinforced concrete.

5-09. <u>Hydraulic Instrumentation Shaft Excavations</u>. The two hydraulic instrumentation shafts for San Pedro Creek Tunnel were constructed according to the contractor's approved submittal. The submittal provided for a 12-inch inside diameter, Schedule 40 steel cased shaft as specified.

However, there were a few changes proposed in the procedures. One change was to drill a 24-inch diameter boring rather than the specified 16-inch boring. Also, since the upstream shaft was actually located within San Pedro Creek, a 54-inch diameter surface casing was used as a work caisson through the water, and a 24-inch diameter corrugated metal pipe, C.M.P., was installed as a permanent stick-up above the creek surface.

Both of these shafts were drilled in April 1988 by Beck Foundation Company using a Northwest 5045 crane-type rotary drill rig. One shaft was located near the outlet shaft at tunnel Station 143+00. It was drilled to the 119.2-foot depth, and was backfilled with 2 feet of drill cuttings to provide the permanent casing seating at the 117.2-foot depth. The other shaft was located in the creek channel near the inlet shaft at Station 199+81. Its drilled depth was 107.0 feet with permanent casing set 2 feet higher on backfilled drill cuttings.

The general construction procedure was first to drill an oversized hole through the overburden and set temporary casing into the impervious clay shale. The remainder of the shaft was then augured at a 24-inch diameter to the total depth. The lower 2 feet of the hole was backfilled with drill cuttings to provide a casing seating about 5 inches above the projected tunnel bore. This was followed by the installation of the 12-inch diameter, Schedule 40 steel, permanent casing. The annular space was backfilled with sand-cement grout, and the temporary casing was removed as the grout approached the ground surface.

The upstream shaft, being in the creek channel, had a couple of additional features. To prevent the stream flow from entering the shaft, a temporary 54-inch diameter steel casing was installed to a depth of 3 feet below the channel, and was removed when the construction was completed. Also, a permanent outer casing was installed at the surface to provide a stick-up of about 2 feet above the water level. When the cement backfill had been poured around the 12-inch diameter permanent casing up to the stream channel, a 24-inch diameter, 16 gauge C.M.P. was pressed into the cement to form an outer casing through the water. The annular space between the C.M.P. and the Schedule 40 casing was also backfilled with the cement

No further excavation was required for the intersection of the shaft and tunnel. The TBM cut through the lower portion of the shaft and removed the backfill cuttings. A 12-inch diameter hole was cut through the precast tunnel liner to access the bottom of the shaft. A sona-tube form was secured between the tunnel liner and the shaft casing. The annular space behind the tunnel liner was then filled with pea gravel, and finally grouted around the sona-tube.

5-10. Tunnel Excavation. As discussed in preceding paragraphs, the tunnel was excavated by a modified Robins TBM and supported with a precast concrete segmental liner. The TBM excavated the 5,843-foot long tunnel to a diameter of 26 feet 11 inches. The precast liner, consisting of 6 segments per ring, was insta'led within the TBM tail shield by a circular erector arm located about 38 reet behind the heading. The liner segments were 4 feet wide and 1 foot thick giving the tunnel an inside diameter of 24 feet 4 inches with an outside annular space of 3.5 inches. The liner was primarily supported with pea gravel blown into the annular space and later grouted with 1:1 cement grout (water-cement ratio by volume) about 500 feet or more behind the heading. The specified lines and grades of the excavation were controlled by laser beam instrumentation.

Although the tunnel excavation encountered no major problems, the rate of advance averaged only half of the anticipated 60 feet per day. The work schedule consisted of two 10-hour shifts per day which usually included Saturdays. The largest advance in a day was 106 feet on July 9, 1989, but the average was 30 feet. The average rate was lowered considerably by the 107 workdays required to complete the first 700 feet of tunnel. This slow start was attributed to an initial learning curve for the workers, mechanical problems, the Christmas holiday season, and the typical difficulties of starting tunnel construction on a curve section; the first 600 feet of the tunnel were in a curve. After the first 700 feet, the tunneling progress improved with only occasional delays. These minor delays were generally only a few days in duration, and often due to the contractor's difficulty in keeping the pea gravel and grouting operations in pace with the excavation rate.

This lag in the pea gravel and grout backpacking became a major concern to the Government, since it was the contractor's proposed primary means of providing positive structural support for the precast segmental liner It was essential for safety and the operational longevity of the tunnel to provide a stable circular liner and to secure that liner with a solid, uniformly grouted contact with the surrounding rock. The circularity of the liner had to be preserved to prevent differential pressures developing around the tunnel. The annular void behind the liner had to be completely filled to prevent deterioration of the surrounding clay shale and to create a uniformly structural contact. Therefore, a timely and thorough placement of pea gravel and grout were crucial not only as initial liner support, but also as final liner stabilization. When the contractor became lax in properly executing these essential operations, the Government was obliged to stop the tunnel excavation until the liner erection procedure was brought into full compliance with the approved plan. When pea gravel support was lacking behind the liner and/or when grouting lagged too far behind the excavation, the Government directed the contractor to cease tunnel excavation until these operations were caught up. These cease work orders were issued five times on the following dates: March 2, March 23, April 7, June 16 and June 19, 1989.

The tunnel excavation began on November 7, 1988 with a scheduled completion date of March 7, 1989. The TBM holed-through into the inlet shaft at 06:05 p.m. on July 13, 1989. This was Thursday evening; the contractor worked a partial crew the next day, but no one worked on Saturday or Sunday. Therefore, completion was four days later on Monday, July 17 when the TBM had passed to the back of the shaft (excavating the lower 3.6 feet) and all of the tunnel liner had been set.

See Appendix B for tunneling progress charts.

PART VI

CHARACTER OF FOUNDATION OR TUNNELING MEDIUM

6-01. <u>General.</u> As anticipated, the Taylor Formation proved to be a relatively stable tunneling medium. It was the only rock formation encountered, and it's generally massive character persisted throughout the excavations. The rock was soft enough to readily excavate without blasting, and yet firm enough to stand well in vertical cuts. Minor crown fallout or block settlement occurred in the softer strata due to excessive exposure before the rock was fully supported; however, these were indeed minor and of little construction consequence. Some inevitable stress relief fracturing occurred, but joints and fractures were generally sparse. In short, the San Pedro Creek Tunnel and Shafts were constructed in impermeable, massive, structurally competent, but variably expansive clay based rock.

The following paragraphs summarize the ground conditions encountered in the tunnel and in each shaft.

6-02. Tunnel Foundation or Medium. The Taylor Formation provided a massive, competent, stable rock medium throughout the tunnel, however, the material varied somewhat along the alignment. As the tunnel was excavated upstream from outlet to inlet it passed through successively older strata within the formation. This was due to the .002 upstream grade, a 2 degree southeastward dip of the bedding, and 32 feet of down-to-the-south faulting at about Station 171+50. These strata were identified from youngest to oldest as M-1 through M-5, and as previously discussed in Part III, the stratigraphically lower and older beds increase in carbonate content. The result is that the M-1 and M-2 materials are more clayey and not as strong as the better indurated limy materials of the M-3 through M-5 strata. Therefore, the first 2,892 feet of tunnel, which was on the downthrown side of the fault, encountered lithologically weaker M-1 and M-2 materials, whereas the 2,951 feet of tunnel upstream of the fault encountered the stronger M-3 and M-4 materials.

These stratigraphic changes in the clay to calcium carbonate ratio presented a pronounced material contrast across the fault at Station 171+50, which roughly divides the tunnel length in half. The M-1 and M-2 strata in the downstream half is dark gray, unctuous, massive, soft to moderately soft, variably calcareous, geologically consolidated or slightly indurated clay based rock which with fissility forms a clay shale or otherwise, where nonfissile, could be classified a claystone. This is the weaker material of the formation having an unconfined compressive strength as low as 5 tsf, but normally around 25 tsf. In the upstream half, the M-3 and M-4 strata (also M-5, though it is below the tunnel elevation) is gray to light gray, earthy, massive, moderately soft to moderately hard with occasional hard lenses, very calcareous or limy, well indurated clay based rock which can be called a clay shale where fissile or claystone where nonfissile. Actually, much of this lower portion of the formation has the high carbonate/clay mixture of an indurated marl and could be classified as a marlstone, or an argillaceous limestone where the calcium

carbonate predominates. This is the strongest material of the formation having unconfined compressive strengths normally around 70 tsf.

These material descriptions give the predominant characteristics of the strata. However, it should be noted that stringers of limy shale occur occasionally in the upper strata, and occasional clayer shale layers occur in the lower strata.

The downstream M-1 and M-2 clayey materials tended to deteriorate when subjected to extensive unsupported exposure by slow tunneling progress. Some crown fallout and block settlement occurred, but nothing large or of long term detriment. As tunneling began the TRM moved only 36 fee: in the first 8 days with the result that fallout developed to 2 feet above the crown for the first 7 feet along the alignment. Further, the generally slow progress in much of the downstream tunnel caused the tunnel bore to be unsupported around the TBM for as much as 5 or 6 days before a cut-section would progress back to the tail shield where the liner could be installed. The material was thus exposed for an extended time to desiccation, air-slaking, and the opening of stress relief fractures. Nevertheless, no major fallouts developed. Some block settlement was noted in places on the TBM shield and over the crown of the liner, but no blocks ever fell around the shield into the invert. Although there was some concern that block settlement at times was warping or deflecting the crown of the tail shield downward, the tightness of the shield against the material could also be attributed to the undulatory maneuvering of the 13M. In addition, a thin 1/4 to 1/2-inch layer of compressed ravelings was sometimes noted at the tail shield cut-out section, which indicated that muck cuttings and/or slaking material was falling around the TBM

The massive character of the formation was always obvious in the cut-out section of the tail shield, and it is doubtful that any exceptionally large blocks ever settled out of the crown. The fact that only 15 of 56 borescope holes in this section of tunnel had fractures is indicative of the persistent massiveness of the formation. Also, all but 2 of the borings with fractures were above springline which suggests that these few fractures were stress relief development in the crown through several days of unsupported exposure. (Borescope observations were conducted in 3-foot deep borings, 7 holes per station spaced 45 degrees apart around the tunnel bore, at Stations 143+63, 143+71, 143+79, 142+87, 143+95, 158+39, 153+47, and 158+55)

A concern derived from fallout behind the liner was in the pea gravel and grout backpacking operation. Chunks and ravelings of deteriorated material fallen around the outside of the liner obstructed the thorough placement of pea gravel throughout the annular space. Voids left by the fallout and open joints required a determined effort to insure that all empty space surrounding the liner was filled with pea gravel and/or grout. However, though some secondary grouting was required, final test borings through the liner to 5 feet within the rock indicated that there was good grout penetration and complete rock consolidation about the liner. This was imperative to maintain the longterm integrity of the surrounding ground, and to prevent differential pressures from developing against the tunnel liner.

Both the material strength and the tunneling rate improved in the upstream half of the tunnel. The better indurated limy M-3 and M-4 materials were more resistant to deterioration, and the unsupported exposure time decreased from several days to a day or less as the tunneling operation improved. As a result, there was no fallout or block settlement upstream from the fault at Station 171+50.

6-03. Outlet Shaft Foundation. The outlet shaft was excavated through 27 feet of alluvial overburden, 13 feet of weathered Taylor Formation, and 111 feet of unweathered Taylor Formation. The alluvial overburden included, from the ground surface downward, 4 feet of clay fill, 3 feet of clay, 2 feet of silt, 10 feet of gravelly clay, 2.5 feet of sand, and 5.5 feet of gravel. Ground water at 200 gpm occurred between the top of the sand layer at the 19-foot depth and the weathered clay shale at the 27-foot depth. The weathered clay shale of the Taylor was tan with gray mottling, soft, generally massive with occasional fractures. The unweathered Taylor was gray to dark gray clay shale, predominantly soft to moderately hard in places, massive, variably calcareous, and occasionally jointed or fractured.

Though the Taylor at the outlet shaft was the younger and softer portion of the formation, it was a sound, firm foundation rock. The top of the M-1 stratigraphic marker bed was at about elevation 524; this was at the 115-foot depth or about 10 feet below the crown of the shaft-to-tunnel transition. Nevertheless, all of the rock formation throughout the shaft was the uppermost Taylor, and essentially the same characteristic material. This was the more clayey and less limy material of the downstream tunnel alignment, which excavated easily while standing very well in vertical cuts. There were a couple of fallout slabs from the crown of the horizontal transition section; the dimensions of one was 6 feet by 4 feet by 2 feet and the other was 6 feet by 3 feet by 1/2 foot. These fallouts were derived from stress relief partings along bedding planes due to a delay in shotcrete applications. Normally, excavation surfaces were smooth showing little disturbance to the in situ character of the material.

The formation was typically massive with only occasional fractures or joints. A few irregular discontinuous fractures were noted in the weathered clay shale in the northern half of the shaft. One nearly horizontal, relatively tight joint striking east and dipping 1 degree north was mapped at elevation 578, the 61-foot depth. Several nearly horizontal bedding plane partings were noted around the 100-foot depth, between elevations 539 and 535. Another essentially horizontal joint opened at the top of a 1-inch thick, white bentonite seam at the 107.5-foot depth, elevation 531.5. Several fractures developed in the lower 12 feet of the transition with apparent dips to the northeast and southwest at 1 to 10 degrees.

6-04. <u>Inlet Shaft Foundation</u>. Being on the upthrown side of the tunnel fault, the inlet shaft extended through four of the stratigraphic units identified within the Taylor Formation. The excavation began in the M-1 material at elevation 623, and the M-2 bed was 16 feet lower at elevation 607 The M-3 and M-4 beds began at elevations 583 and 514 ,respectively, the 40-and 109-foot depths. It is noteworthy that a moderately hard to hard limy shale layer was located at the top of each of these stratigraphic units, these are

the marker beds which are distinguishable on electric logs. Throughout the shaft the material was true to the character of each stratigraphic unit, progressing downward from soft clayey rock in the upper strata to harder, limy, well indurated rock at depth.

The entire inlet shaft is constructed within unweathered Taylor Formation, since the overlying weathered and alluvial materials were removed previously during the approach channel work. The upper 62 feet of the shaft is in mostly soft strata which excavated smoothly with little disturbance to the in situ formation. However, the harder rock in the lower 57 feet of the shaft required percussion excavation by hydraulic ram. Although the excavation was controlled somewhat by indistinct horizontal bedding, the material tended to break in conchoidal, angular patterns giving a slightly rough texture to the excavated surface. Tight, thin, discontinuous, and apparently shallow fractures developed along the nearly horizontal bedding planes between elevations 532 and 525, the 91-to 98-foot depths. The formation stood well throughout the shaft excavation, and the well indurated rock in the lower shaft stood extremely well in both vertical and horizontal cuts. The increased carbonate to clay ratio of these lower strata made the rock harder and more brittle, but also less susceptible to desiccation, air slaking and sloughing.

The formation was persistently massive. There were relatively few fractures, and no major joints that extended completely through the shaft. A few irregular discontinuous fractures were located in the upper 3 feet of the southern half of the shaft. Some nearly horizontal discontinuous fracturing was noted at elevation 608, the 15-foot depth. A nearly horizontal, calcite healed joint was noted at elevation 584, the 39-foot depth, but did not extend through the N-NE quadrant of the shaft. In the shaft's S-SW quadrant, between elevations 539 and 536, there were two discontinuous joints with apparent dips of 2 degrees and 3 degrees SE. The few other fractures were thin, tight, short, and probably shallow breakage planes caused by the percussion excavation.

6-05. Maintenance Shaft Foundation. The maintenance shaft being located at tunnel Station 181+77 is on the upthrown side of the fault at Station 171+50. Therefore, like the inlet shaft, it extends from the softer, clayey M-1 strata near the surface into the harder, more limy, and better indurated materials at depth. The maintenance shaft, however, does not extend beyond the M-3 strata, since it is structurally down-dip from the inlet The top of the M-2 strata is at elevation 582.5, the 60-foot depth, and the top of the M-3 is at elevation 562.5, the 80-foot depth. The top of the M-4 strata is correlated at elevation 501, which would be 19.5 feet below the shaft

The raintenance shaft is constructed through 16.0 feet of overburden and 106 feet of Taylor Formation. From surface elevation 642 5, it extends progressively downward through 7.3 feet of clay fill, 8.7 feet of clay, 17 8 feet of weathered clay shale, 8 7 feet of partially weathered clay shale, and 79.5 feet of unweathered clay shale. Due to the impermeable character of both the overburden and the primary formation, there is no observable ground water other than sparse wetness in the overburden. The upper 44 feet of weathered and unweathered clay shale is soft, clayey M-1 material Below the 60-foot

depth, the M-2 strata becomes interbedded with moderately hard limy layers of several inches thickness. A 6-foot thick limy bed occurs at the 70-foot depth. Below the M-3 contact at the 80-foot depth the material is generally moderately soft to moderately hard, more limy, and well indurated. A moderately hard, very limy zone extends between the 90 and 95-foot depths.

The Taylor Formation at the maintenance shaft was massive below the 42-foot depth. The weathered and partially 'eathered material of the upper shaft was fractured with an average spacing of about 5 feet, although it varied from less than 1 foot to 12 feet. Except for nearly horizontal joints at the 40 and 42-foot depths, the fractures were mostly high angle. Those which occurred in the largely unweathered material below the 33.8-foot depth were channels for chemical weathering; weathering had oxidized the shale from gray to reddish tan in 1 to 2-inch wide bands along the fractures. Since the shaft was excavated by drilling and reaming, the scraping of the reaming blades along the shaft walls may have obscured occasional tight fractures or joints in the lower shaft. However, the formation appeared unfractured below the 42-foot depth, and no sloughing occurred.

6-06. <u>Vent Shaft Foundations</u>. The two vent shafts for San Pedro Creek Tunnel were drilled on each side of the fault at Station 171+50. The vent shaft near Durango Street is located at Station 158+14 on the downthrown side of the fault, and in the soft, clayey Upper Taylor Formation. The top of the M-1 strata is at elevation 529.3, the 110-foot depth or 11 feet above the bottom of the shaft. The vent shaft near Salinas Street is located at Station 185+74 on the upthrown side of the fault, and extends from the softer, clayey M-1 strata into the harder limy M-3 materials. The top of the M-2 strata is at elevation 601, the 42-foot depth, and the top of the M-3 is at elevation 578, the 65-foot depth. The top of the M-4 strata is correlated at elevation 510, the 133-foot depth or 16 feet below the bottom of the shaft.

The Durango Street vent shaft at Station 158+14 extends through 23.0 feet of overburden, 16.8 feet of weathered Taylor Formation, and 81.2 feet of unweathered Taylor Formation. Progressively downward, the overburden includes 1.5 feet of sand fill, 10.5 feet of gravelly clay, and 11.0 feet of clayey gravel. Free water was encountered between the 16.0 and 23.0-foot depths when the shaft was drilled in May 1988. The weathered Taylor consists of soft, fractured clay shale. The unweathered Taylor is soft, massive, variably calcareous clay shale. The formation stood well with no sloughing during the shaft sinking.

The Salinas Street vent shaft at Station 185+74 extends through 13.58 feet of overburden, 17.42 feet of weathered Taylor Formation, and 86.0 feet of unweathered Taylor Formation. The overburden consists of 2.5 feet of gravelly clay fill overlying 11.08 feet of clay. During the shaft sinking in May 1988, only a small amount of ground water flowed along a joint at the 13.0-foot depth and along the formation contact at the 13.58-foot depth. The weathered Taylor is soft, fractured clay shale at this shaft also. The unweathered Taylor begins in the soft, clayey M-1 strata, but is increasingly moderately soft and more calcareous in the M-2 below the 42-foot depth. Below the 65-foot depth, the M-3 strata is moderately soft to moderately hard with

highly calcareous or limy layers. The formation was massive and stood well; no sloughing occurred during construction.

6-07. Hydraulic Instrumentation Shaft Foundations. Like the vent shafts, the two hydraulic instrumentation shafts were drilled on each side of the fault at Station 171+50. One of the shafts is located near the outlet at Station 143+00. It is on the downthrown side of the fault, and is in the soft, clayey Upper Taylor Formation. The top of the M-1 strata is at elevation 522, the 116-foot depth, or 3.2 feet above the bottom of the shaft. The other shaft is at Station 199+81 near the inlet, and is on the upthrown side of the fault. It begins in the soft, clayey M-1 strata, and extends well into the harder, limy M-3 strata. The top of the M-2 strata is at elevation 605, the 30.8-foot depth, and the top of the M-3 is at elevation 580.8, the 55.0-foot depth. The M-4 is correlated at elevation 512, the 123.8-foot depth, or 16.8 feet below the bottom of this shaft.

The hydraulic instrumentation shaft near the outlet extends through 27.0 feet of overburden, 12.6 feet of weathered Taylor Formation, and 79.6 feet of unweathered Taylor Formation. From the ground surface downward, the overburden consists of 2 feet of clay fill, 10 feet of clay, 8 feet of gravelly clay, and 7 feet of clayey gravel. The lower 7 feet of clayey gravel contained free water during construction in April 1988. The weathered Taylor Formation is soft, fractured clay shale. The unweathered Taylor is soft, massive, variably calcareous clay shale, but has occasional thin, moderately soft to moderately hard, highly calcareous layers. The formation stood well without sloughing during construction.

The hydraulic instrumentation shaft near the inlet extends through 0.3 feet of concrete channel liner in San Pedro Creek, 11 feet of weathered Taylor Formation, and 95.2 feet of unweathered Taylor Formation. There is no alluvial overburden at this site since chan-el improvements to the creek have placed concrete liner directly on weathered clay shale of the Taylor Formation. The weathered clay shale is soft and somewhat blocky, but with little indication of fracturing. During the drilling in April 1988, a trace of free water was observed at the 5-foot depth; this could have been seepage around the 3-foot deep surface caisson or ground water flow along a formation fracture. The unweathered Taylor Formation is soft, clayey M-1 and M-2 strata in the upper 55.0 feet of the shaft. Moderately soft to moderately hard, limy zones increase with depth through the underlying M-3 strata. The formation was massive and stable with no sloughing during construction.

PART VII

FOUNDATION TREATMENT

- 7-01. <u>General</u>. There was no major foundation treatment required for the tunnel or shafts. However, two of the support procedures may also be considered methods of foundation treatment. These two operations were the rock anchor installations in the shafts and the grouting of the tunnel liner. Although both the rock anchors and the grouting were required as part of the excavation support, they may also be considered foundation treatment in that they enhanced the in situ stability of the rock formation. These operations have been described as support procedures in Part V, but will be further discussed in this section.
- 7-02. Rock Anchors. There were three general types of rock anchors used on the San Pedro Creek project. Type I and Type II rock anchors were used in the outlet shaft. Type I and Type III rock anchors were used in the inlet shaft. The type differences consisted of variations in length and corresponding bonding capacities. The rock anchors were normally stressed to design loads and then locked off at 80¢ of that load, which varied with the length of the rock anchor. Type I rock anchors were 18 feet long, had a design load of 90 kips, and a lock-off load of 72 kips. Type III rock anchors were 21 feet long, had a design load of 110 kips, and a lock-off load of 88 kips. Type III rock anchors were 15 feet long, had a design load of 100 kips, and a lock-off load of 80 kips. The Type III anchors were used exclusively in the better indurated rock at the inlet shaft, and thus had a higher bonding capacity for the shorter length of anchor.

All three types of rock anchors were similar in materials and construction. They were all 1.25-inch diameter, No. 10 Dywidag threadbars, and were cement grouted into 5-inch diameter holes. The anchor grout was a non-corrosive expansive admixture with a minimum 28 day compressive strength of 3000 psi. The recommended pumping pressure for the grout was 30 psi. PVC spacers were used at equal distances along the boring to keep the anchor in the center of the hole. A 2-inch thick, 5-inch diameter styrofoam donut was placed around the anchors at the 1.0 to 1.5-foot depth to act as a grout barrier; the styrofoam was also supposed to provide a compressible cushion which would allow the anchor bar to move if the bonding capacity was exceeded during the stress loading. The outer foot or so of hole beyond the styrofoam donut was backfilled with dry-pack cement around a PVC bond breaker covering the anchor bar. An 8 to 10 inch square, 1.5-inch thick Dywidag bearing plate was installed against the shotcreted shaft surface at the outer end of the anchor bar.

The design of these rock anchors provided a support effect similar in principal to "soil nails" rather than typical rock bolts. Soil nails are normally relatively short steel bars of a fully bonded length installed as reinforcing inclusions to the in situ ground. Usually closely spaced, they produce a zone of reinforced ground which performs in a manner similar to a retaining wall. Soil nails are not stressed although it is common to apply a

small seating load. Unlike soil nails, rock bolts are stressed after installation with the load transferred along a distal, fixed anchorage length; this distal anchorage binds the unbonded outer rock to the more stable ground mass at depth. The rock anchors in the outlet and inlet shafts were stressed like rock bolts, and yet, like soil nails, they were bonded for nearly their entire length. Only the outer 1.0 to 1.5 feet of bar length was unbonded. Considering the thickness of shotcrete on the shaft wall, this left only the outer few inches to 1.0 foot of rock unbonded, and the stressing load was distributed along the rest of the bar. Therefore, the rock anchors acted as stress loaded soil nails rather than bolts anchored at depth.

Although this type of rock anchor appeared to work satisfactorily in the massive rock of the San Pedro Creek shafts, there is some doubt that the anchors could actually sustain their submitted design load. Load tests on instrumentation rock bolts in the soft clayey rock at San Pedro Creek Outlet Shaft had difficulty in achieving a required 20 kips test load. It was therefore questionable that 90 to 110 kips were achieved on support rock anchors at this same shaft.

The contractor's method of stressing the support rock anchors was somewhat dubious. The method of stressing placed the jacking frame against the bearing plate which covered the grouted anchor hole and rested on the shotcreted shaft wall. When a load was applied to the jacking frame, the bearing plate restrained the grout column from moving. Thus, the bond between the grout and rock could not break if its normal load capacity was exceeded. If failure occurred it would have been at the relatively strong bond between the anchor bar and the grout column. It was intended that the 2-inch thick styrofoam donut would collapse enough to allow failure of the grout-to-rock bond However, this was rather speculative since the strength of the styrofoam under complete confinement was unknown.

In any case, these rock anchor "nails" apparently provided an effective reinforcement in the massive rock at the San Pedro Creek shafts, and no support problems developed. However, in jointed, more thinly stratified, blocky ground as at the San Antonio River Outlet Shaft these anchor nails would be less effective than the longer typical rock bolts having a distal anchorage at depth. This is more relative to the forthcoming foundation report on the San Antonio River Tunnel. Nevertheless, it is significant to mention that random failure and creep tests performed on Type I rock anchors at that outlet revealed load capacities of only 16 to 38 kips in similar soft, clayey rock. As a result, the anchor loading procedure was eventually changed to use a jacking frame large enough to straddle the bearing plate. This left the grout column free to move if the anchor failed.

Although three general types of rock anchors were used for the most part in the San Pedro Creek shafts, there was actually a fourth type. This fourth type of anchor was used in only two places at the inlet shaft, and was not a major design requirement. There were 24 of these resin grouted, 8-foot long, 3/4-inch diameter anchors installed at elevations 617 and 612 in the upper 21 feet of the shaft. These small anchors were only a precautionary measure auded to the support design, which required only 3 inches of shotcrete for the upper shaft

7-03. Tunnel Liner Grouting. Grouting of the annular space between the tunnel liner and the surrounding rock was primarily to establish a solid contact between the liner and the rock, but it also consolidated the surrounding rock by filling open fractures, joints, and occasional elongated voids left by minor block settlements in the crown. Grouting behind tunnel liners is usually called backpack grouting, and is largely for support. The grouting of fissures and voids in the loosened rock surrounding tunnels is referred to as consolidation grouting, and is predominantly a stabilization treatment. Consolidation grouting often requires the drilling of grout holes to the depth of formation disturbance to provide a full distribution of the grout. However, the zone of disturbance in the massive material surrounding the San Pedro Creek Tunnel was only several feet thick. Therefore, backpack grouting and consolidation grouting were effectively accomplished in the same operation as the grout pumped behind the liner also penetrated well into the relatively shallow fractures.

The grouting procedure proved to be reasonably thorough although it was done in a patchwork fashion. The procedure was to grout in horizontal strips at various locations with a general upward progression from the invert holes. Two 2-inch diameter grout holes were constructed in each liner segment which allowed the upper holes to provide venting and observation ports. Injection holes were moved vertically and horizontally beyond holes which were plugged due to previous grout flows. Adjoining grout sections would overlap previous grouting, or upstream grouting sections would merge with advancing downstream sections. Grouting at the crown required sustained pumping at gravity flow until pressure could be obtained or a secondary grouting which could maintain pressure. This method eventually produced a forward slope of grout from a downstream injection point in the crown to an upstream edge in the invert, covering approximately 200 feet of alignment. The grout was a 1:1 cement to water ratio by volume, and was pumped at a maximum pressure of 28 psi.

Quantitative data on the pea gravel and grout placement shows that the primary backfilling extended well around the liner into the crown annular space (See Appendix C). The volume of the 3.5-inch wide annular space was calculated to be 98 cubic feet per 4-foot liner ring; however, it should be noted that part of this void was no doubt filled with rock cuttings or rubble in places. A pea gravel density of 95 pounds per cubic foot was used to compute the amount of pea gravel backfilled behind the rings, which averaged 46 cubic feet per ring. The average placement of grout per ring was estimated at 55 cubic feet. The pea gravel volume included approximately 40% voids which would consume part of the grout placement. Therefore, of the 98 cubic feet of annulus behind each ring, 46 cubic feet was filled with pea gravel and 37 cubic feet was filled with grout. This gave an average of 83 cubic feet of backfilled pea gravel and grout which was 85% of the annular space. Since much of the invert liner was placed directly on the excavated surface, most of the void was in the crown rather than arranged concentrically into a 3.5-inch wide annular space. Thus, the 85% backfill would extend well into the crown area after the primary pass of grouting.

The 85% backfill estimate may be considered a best case scenario since it is based on bulk placement quantities and ignores material wastage On the other hand, this wastage would be partially offset or possibly exceeded in places by

the volume of rock settlement and ravelings. Also the amount of grout required to fill the pea gravel voids is somewhat speculative and subject to variables such as the presence of extraneous moisture and granular fines. In any case, the remaining annular space was filled by secondary pressure grouting conducted in crown borings spaced on 50-foot centers along the tunnel alignment.

A core sampling investigation by the Government determined that the grouting operation had effectively accomplished both the backpacking of the tunnel annular space and the consolidation of the surrounding rock. Three 100-foot long test sections were established at Stations 148+94 to 149+94, 169+98 to 170+98, and 189+82 to 190+82. Core sampling was conducted through the liner grout holes at every fifth ring in each test section. Four core samples were taken at each test ring, one in each quadrant, with a rotational sequence from ring to ring so that all 12 grout hole positions were sampled in each 100-foot test section. Core sampling was extended to the depth of solid, undisturbed rock in each hole, which was generally within 5 feet of the tunnel bore. Also, to determine the effectiveness of consolidation grouting in a reach of known rock settlement, core samples were taken every 50 feet in the crown between Stations 142+87 and 148+55. Eight other borings sampled beneath the invert liner between Stations 166+22 and 179+82. The findings of all 93 core borings indicated that grout had penetrated well into all fractures surrounding the tunnel bore and, except where liner segments rested directly on the excavated surface, the annular space had been completely filled.

See Appendix C for grouting data.

PART VIII

CONSTRUCTION MATERIALS

The earth materials used in the Phase II tunnel construction consisted of pea gravel and concrete aggregate. These materials were obtained from local San Antonio suppliers. The pea gravel used as tunnel liner backfill was supplied by Capitol Aggregates, Inc. at 11551 Nacogdoches. Cast-in-place and backfill concrete was obtained from Pioneer Concrete of Texas, Inc. at 15080 Tradesmen, and contained aggregate supplied by Redland Worth Corporation located at 17910 IH-10 West. The concrete for the precast liner segments, manufactured by Sehulster Corporation at 7386 Grissom Road, was supplied by Meader Construction Company, Inc., whose plant was nearby at 7510 Grissom Road. Aggregate for the Meader concrete was provided at first by Redland Worth Corporation, but later by Vulcan Materials Company. The Vulcan Materials Office was at 800 Isom Road, however the aggregate came from a limestone quarry on Huebner Road relatively close to the precast plant. Concrete aggregate analyses were included in the mix design submittals which were reviewed and approved by the Government.

GEOTECHNICAL INSTRUMENTATION

9-01. General. The contract specifications provided for a geotechnical instrumentation program to monitor ground behavior at the outlet shaft and at two designated stations in San Pedro Creek Tunnel. The Contractor, Ohbayashi Corporation, retained the services of Woodward-Clyde Consultants to implement the program, and their Final Instrumentation Report is included as Appendix G of this report. The instrumentation was designed to monitor any ground movements and/or stress developments around the excavations with the intent to provide data for safety observations, design verification, and future design applications. Immediate notification of the Government was required during construction when ground movements exceeded 0.25 inches, or when stresses exceeded 5 kips (34.7 psi) in the outlet shaft, or when stresses greater than 5 tsf (69.4 psi) were indicated in the tunnel. These parameters were not exceeded in the outlet shaft. However, they were exceeded to some extent in the tunnel, but this was considered the localized effects of the tunneling operations. A detailed discussion and interpretation of the instrumentation data can be found in Appendix G. The following paragraphs will describe each instrument installation.

9-02. <u>Outlet Shaft Instrumentation</u>. The outlet shaft instrumentation consisted of 3-position extensometers, rockbolt load cells, and total pressure load cells designated for installation at three elevations -- 604 (35-foot depth), 575 (64-foot depth), and 550 (89-foot depth). However, since bonding rock anchors would be difficult in the clay-like weathered shale at elevation 604, the rockbolt load cells planned for that elevation were installed in unweathered clay shale at elevation 596. Also, to accommodate concrete pours for the permanent shaft liner, the total pressure load cells planned for installation in pairs at elevations 604, 575, and 550 were actually all installed at elevation 564.

Four, 26-foot long, multiple position borehole extensometers (MPBX) were installed horizontally and 90 degrees apart at each of the three designated elevations -- 604, 575, and 550. These were 3-position MPBXs having three measurement rods anchored successively at depths of 3 feet, 11 feet, and 26 feet. The rods were cement grouted into a 27-foot deep, 3-inch diameter borehole. The outer ends of the rods were encased in an electrical sensor head installed in a 1-foot diameter by 2-foot long blockout in the shaft wall. These instruments were designed to measure any horizontal movements in the surrounding ground.

Four, 1-inch diameter, 39.5-foot long rockbolts with load cells (RBLC) were installed horizontally and 90 degrees apart at each of three elevations -- 596, 575, 550. These installations were offset 45 degrees from the MPBX locations. The back 19 feet of the rockbolt was anchored with a two-component resin grout in a 1 5/8-inch diameter hole. The outer 20 feet of the bolt was unbonded in a 3-inch diameter section of the boring, this part of the bolt was wrapped with two layers of bituminous tape and covered for 18 fee; with 2-inch

diameter PVC pipe. The outer 6 inches of the bolt extended through a 1-inch thick steel bearing plate into a 1-foot diameter blockout cut into the outer foot of the shaft wall. This outer end of the bolt was mounted with a load cell which was wired for electronic readings and secured with an outer seating nut. The purpose of the RBLCs was to detect rock loads or stresses developing in the shaft walls.

In addition to the MPBXs and RBLCs installed in the surrounding rock, total pressure load cells were installed at the outer edge of the permanent concrete liner. Whereas the other instruments detected movements and loads within the ground, the total pressure load cells were designed to determine stress developments on and within the permanent liner. There were 6 total pressure load cells installed at elevation 564 (75-foot depth) on 60 degree spacings.

9-03. <u>Tunnel Instrumentation</u>. The tunnel instrumentation was designated for installation at Station 143+75 and Station 158+47. The instrumentation specified for each station consisted of a 6-position MPBX, one RBLC, three total pressure load cells, three reinforced concrete strain meters, and eight tape extensometer eye bolts. In addition, 18 survey reference/displacement markers were installed on the ground surface between Stations 143+00 and 145+00. Also included with the instrumentation program were 56, 8 foot deep borescope observation holes drilled adjacent to the instrument stations.

A 6-position MPBX was installed in a surface boring above the tunnel at each of the two instrument stations. These MPBXs had six measurement rods cement grouted into 3-inch diameter borings which extended to within three feet of the tunnel crown. The rods were anchored at depths of 60, 80, 89, 99, 107, and 111 feet in a 113-foot deep boring at Station 143+75, and at depths of 64, 84, 94, 104, 111, and 116 feet in a 117-foot deep boring at Station 158+47. The upper ends of the rods were encased in an electrical sensor head installed in a 10-inch diameter by 3.0-foot deep manhole. The purpose of these MPBXs were to measure any vertical movements over the tunnel excavation.

A 1-inch diameter, 39.5-foot long rockbolt with load cell was specified for both tunnel instrumentation stations. These RBLCs were constructed in the same manner as those described for the outlet shaft except for a few changes at the Station 158+47 installation. This RBLC was 45 feet long, and had a 5-inch diameter boring with 25 feet of cement grout anchorage. Both RBLCs were installed through the tunnel liner at about 15 degrees west of the crown center line. Like those in the outlet shaft, these instruments were intended to detect rock loads or stresses developing in the tunnel wall.

Three total pressure load cells and three reinforced concrete strain meters were installed at each instrumentation station. These instruments were installed on a 120 degree spacing around the tunnel liner with a 2-foot offset from the center line. At each location a total pressure load cell was installed in a blockout at the back of the liner with a reinforced concrete strain meter embedded within the liner concrete at the same position. The purpose of these instruments was to detect load distributions and stress developments on and within the liner.

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Tape extensometer eye bolts were installed at both instrumentation stations for liner convergence measurements between opposing reference points. There were 8 reference points at each station spaced from the center line at 45 degree intervals. At Station 143+75 an eye bolt was installed at each of the 8 reference points; however, only 6 eye bolts were installed at Station 158+47, since the fan line and muck hauling tracks blocked the crown and invert points respectively.

Although no measurable surface movements were anticipated or actually occurred, survey reference points were established on the ground surface above the tunnel to document that such expectations were valid. There were 18 survey reference points established between tunnel Stations 143+00 and Station 145+00. The top of the hydraulic instrumentation shaft at Station 143+00 and the top of the MPBX at Station 143+75 were established as two of the points. Monument spikes having 3/8-inch square shanks were used for 6 of the points which were located on an asphalt paved driveway. The remaining 10 survey points were in unpaved areas and consisted of a 3/4-inch bar, 4 feet long, driven to flush with the ground surface.

Borescope observations were made in seven 8-foot deep by 3-inch diameter borings cored at each of eight stations along the tunnel alignment. The first set of 35 borings were drilled at Stations 143+63, 143+71, 143+79, 143+87, and 158+55. The second set of 21 borings were drilled at Station 158+39, 158+47, and 158+55. The borings were cored by a Watson drilling rig mounted on the liner erector arm at the back of the tail shield. The erector arm rotated the drill to each of the seven circumferential locations which were on 45 degree spacings with the centerline invert position omitted. The borings were drilled through holes cut in the tail shield and in the cut away shield section below springline. The tunnel face was about 37 feet upstream from the borescope holes during the investigations. Both sets of borings were viewed by Woodward-Clyde and Government representatives with a borescope, and phocographs were taken of fractures with a 35 mm camera attached to the borescope. The second set of borings near instrument Station 158+47 was viewed by the Government with a down-hole video camera.

The first set of 35 borings between Station 143+63 and 143+95 were drilled and investigated between December 23, 1988 and January 5, 1989. Fractures were found in 10 of the 35 borings. The fractures ranged in width from 0.02 inches to 1.6 inches and were mostly along the nearly horizontal bedding planes. All fractures but one were above springline.

The second set of 21 borings between Stations 158+43 and 158+59 were drilled and investigated from March 20 through March 22, 1989 Fractures were observed in 5 of the 21 borings. The fractures ranged in thickness from 0 025 to 1.5 inches, and were mostly along the nearly horizontal bedding planes. Fractures were observed in all three crown borings, in one springline boring, and in only one boring below springline.

See Woodward-Clyde's illustrations, data plots, and detailed evaluation of this instrumentation program in Appendix ${\tt G}.$

FOURDATION PROBLEM AREAS

The Taylor Formation proved to be a competent tunneling medium, and should cause no future problems. The clay shale was soft enough to excavate easily, and yet, stood well throughout most of the excavations. There were some minor crown fallouts or block settlements in the downstream half of the alignment, particularly in the first 700 feet from the outlet where the tunneling rate was slow. However, the surrounding rock and annular pea gravel have been well consolicated by grouting. This gives a solid, uniform radial contact between the ground and the tunnel liner to insure that no differential pressures develop. The tunnel liner has been designed for potentially high radial swell pressures, and no problems are anticipated.

Due to the expansive nature of the clay shale in places, an effort was made to keep the excavated surfaces dry to prevent moisture induced swelling. However, it was inevitable that some of the rock would be exposed to water from grout bleed-off or unforeseen spillage. There were two particular places along the alignment where the formation was notably wettel, Station 166+22 to 179+82 and in the vicinity of Station 178+49.

The section of tunnel rock between Stations 166+22 and 179+82 was not subjected to a large quantity of water, but remained damp or wet through most of the construction period. The night shift crew during excavation reported that the TBM seemed to hit something hard in this area, and it was speculated that the cutterhead may have clipped an artesian water well that barely intersected the tunnel bore. However, no sign of a well was observed in the tail shield cut-away section below springline. It is rather plausible that the wetness observed in this area was bleed-off water from the grouting operation. Seepage never flowed at any appreciable rate, but wetness would coze from small water accumulations in the liner grout holes. This slow seepage could well have been grout bleed-off water trapped behind the liner in a pea gravel pocket between grouting zones. The area was grouted twice to insure that all formation fractures were sealed; there was no grout take in the second attempt which indicated that the rock and liner annular space were tight. Although this reach of tunnel rock was wetted for an extended time, the ultimate development of swelling pressure will depend on the expansive clay content at this particular location and the amount of swell dissipation into available openings. In any case, the tunnel liner is designed to support the anticipated swell pressures.

Another section where the tunnel rock was subjected to notable wetting was a reach extending approximately 200 feet to each side of Station 178449. An abandoned artesian water well was severed by the TBM at Station 178449. The ground was exposed to well leakage until the liner annular space was grouted past the site after the TBM had proceeded 200 feet beyond it. This delay was necessary to allow the TBM trailing gear to pass. The partially sealed well leaked only about 2 gallons per minute, but proved quite difficult for the contractor to plug (see Section 3-02, e). When the liner annular space was

grouted, a hole was left in the liner for a well outlet. Therefore, the well continued to leak inside the tunnel liner for about 5 months, and some seepage no doubt migrated along rock fractures behind the liner. The well was eventually plugged and the surrounding ground grouted, but the rock formation in this area had been subjected to considerable wetting. Nevertheless, little or no swelling is expected in this section because it is in the lower more calcareous portion of the formation; this usually means a correspondingly lower clay fraction with less significant amounts of expansive clay minerals. Also, as stated above, the tunnel liner is designed to support anticipated swell pressures.

There were no serious problems with the geologic conditions encountered by the tunnel and shaft excavations. The notable occurrences mentioned above are documented herein only as information which may have some unforeseen significance at a later date. No future foundation problems are anticipated.

RECORD OF FOUNDATION INSPECTIONS AND GEOLOGIC DOCUMENTATION

Rock exposures in all shaft excavations were inspected, mapped or logged, and photographed by a geologist. The excavated tunnel bore was observed periodically by the geologist at the tail shield cut-away section below springline. However, no attempt was made to map the tunnel rock due to incomplete exposure, congested working area, and the generally massive, largely featureless character of the rock in this area. The formation was generally soft to moderately hard, massive rock that excavated smoothly and presented few difficulties. The following is a list of mapping and logging dates during the excavation of each shaft:

Feature	Date		Depth Mapped		terval Logged	Geologist
Hydraulic Inst. Shaft SP-1	25 API	88	Logged	to	119.2	R. Burns
Hydraulic Inst. Shaft SP-5	28 API	R 88	Logged	to	107.0	R. Burns
Vent Shaft SP-4	2-4 MA	¥ 88	Logged	to	117 0	R. Crutchfield
Vent Shaft SP-2	11-13 MA	88 Y	Logged	to	121.0	R. Crutchfield
Maintenance Shaft SP-3	9 MAY-11 AU	G 88	Logged	to	122.0	Burns-Crutchfield
Outlet Shaft (Mapped)	29 JA 9 FE 12 FE 16 FE 19 FE 23 FE 29 FE 2 MA 16 MA 25 MA 29 MA 1 AP 5 AP 8 AP 13 AP	B 88 88 88 88 88 88 88 88 88 88 88 88 88	0.0 12.0 16.0 20.0 24.0 28.0 32 0 37 0 41.0 45.0 50 0 60.0 63.0	to	16.0 20.0 24.0 28.0 32.0 37.0 41.0 45.0 50.0 60.0 63.0 67.0	R. Crutchfield R. Crutchfield R. Crutchfield R. Crutchfield Burns-Crutchfield
	15 AP		72.5	to		Burns-Crutchfield

Feature	Date	Depth Interval Mapped or Logged	Ceologist
Outlet Shaft	20 APR 88	77.0 to 82.0	Burns-Crutchfield
(mapped) cont.	22 APR 88	82.0 to 85.0	Burns-Crutchfield
	5 MAY 88	85.0 to 91.0	Burns-Crutchfield
	11 MAY 88	91.0 to 95.0	Burns-Crutchfield
	17 MAY 88	95.0 to 99.0	R. Crutchfield
	26 MAY 88	99.0 to 107.0	R. Burns
	6 JUN 88	107.0 to 111.0	Burns-Crutchfield
	13 JUN 88	111.0 to 117.0	Burns-Crutchfield
	1 JUL 88	117.0 tc 122.0	R. Burns
	8 JUL 88	122.0 to 126.0	R. Crutchfield
	23 JUL 88	126.0 to 131.0	R. Crutchfield
	29 JUL 88	131.0 to 135.0	R. Crutchfield
	4 AUG 88	135.0 to 140.0	R. Crutchfield
	8 AUG 88	140.0 to 146.0	R. Crutchfield
Inlet Shaft	11 OCT 88	0.0 to 4.0	R Crutchfield
(mapped)	13 OCT 88	4.0 to 13.0	Crutchfield
	15 OCT 88	13.0 to 17.0	R. Crutchfield
•	18 OCT 88	17.0 to 21.0	R. Crutchfield
	6 JAN 89	21.0 to 25.0	R Crutchfield
	17 JAN 88	25.0 to 30.0	R. Crutchfield
	20 JAN 88	30.0 to 34.0	R. Crutchfield
	25 JAN 89	34.0 to 40.0	R. Crutchfield
	2 FEB 89	40.0 to 45.0	R. Crutchfield
	9 FEB 89	45.0 to 53.0	R. Crutchfield
	15 FEB 89	53.0 to 62.0	R. Crutchfield
	23 FEB 89	62.0 to 67.0	R. Crutchfield
	7 MAR 89	67.0 to 71.0	R Crutchfield
	23 MAR 89	71.0 to 79.0	R. Crutchfield
	31 MAR 89	79.0 to 87.0	R Crutchfield
	20 APR 89	87.0 to 92.0	R. Crutchfield
	2 MAY 89	92.0 to 96.0	R. Crutchfield
	16 MAY 89	96.0 to 100.0	R. Crutchfield
	31 MAY 89	100.0 to 103.0	R. Crutchfield
	19 JUN 89	103.0 to 115 0	R Crutchfield
TBM Hole-through	13 JUL 89	115.0 to 118.6	R. Crutchfield

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APPENDIX A

Photographs



View northeast across initial 12-foot deep excavation of San Pedro Creek Outlet Shaft. Rib and lagging collar in left background.

29 Jan 88 San Pedro Crk Tunnel Photo No. 1



View northeast into initial excavation for San Pedro Creek Outlet Shaft.

29 Jan 88 San Pedro Crk Tunnel Photo No. 2

EXHIBIT 1



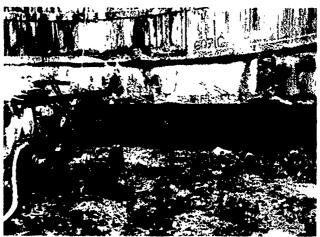
View southwest into San Pedro Crk Outlet Shaft to elevation 622, 17-foot depth.



View southwest into San Pedro Cak Outlet Shaft to elevation 618. 21-foot depth. Alluvial ground-water inflow began at elevation 620.

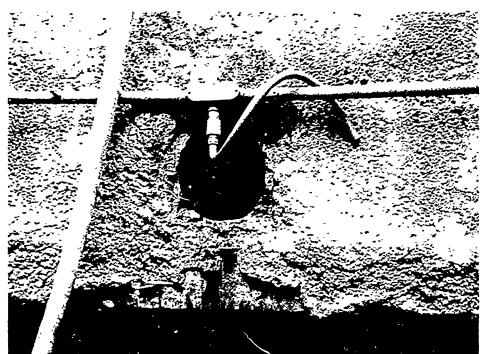
12 Feb 88 San Pedro Crk Tunnel Photo No. 4

View northeast showing weathered clay shale between elevations 611 and 607 (32-foot depth) in San Pedro Crk Outlet Shaft.

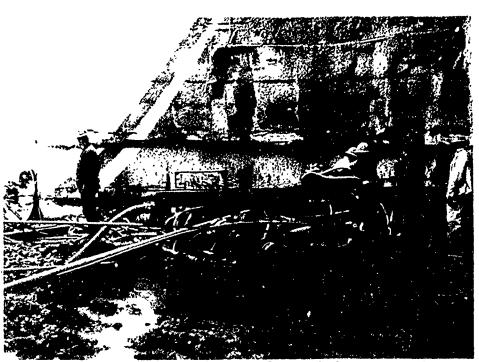


View southeast in San Pedro Crk Outlet Shaft showing the contact between weathered and unweathered clay shale at elevation 599, 40-foot depth.

2 Mar 88 San Pedro Crk Tunnel Photo No. 6

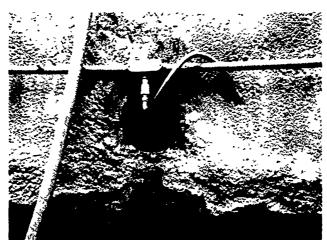


3-position extensometer, MPBX, installed at elevation 604, 35-foot depth, in San Pedro Creek Outlet Shaft.



Drilling hole for installation of rock bolt load cell. RBLC, at elevation 596, 43-foot depth in San Pedro Creek Outlet Shaft.

18 Mar 88 San Pedro Crk Tunnel Photo No. 8



3-position extensometer, MPBX, installed at elevation 604, 35-foot depth, in San Pedro Creek Outlet Shaft.



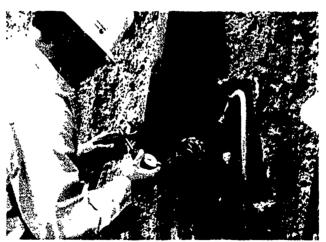
Drilling hole for installation of rock bolt load cell. RBLC, at elevation 596, 43-foot depth in San Pedro Creek Outlet Shaft.

18 Mar 88 San Pedro Crk Tunnel Photo No. 8

EXHIBIT 4



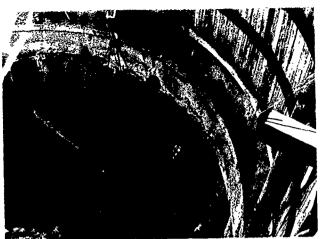
10-Ton pull-out test on rock bolt for load cell at elevation 596, 43-foot depth, in San Pedro Creek Outlet Shaft. Test by Woodward-Clyde Consultants.



Close-up view of gauge to measure rock bolt movement in 10-Ton $\operatorname{pull-out}$ test shown above.

18 Mar 88 San Pedro Crk Tunnel Photo No. 10

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Grouting alluvial aquifer through ldgging in San Pedro Creek Outlet Shaft. Bottom of shaft at elevation 598, 41-foot depth.

9 Mar 88 San Pedro Crk Tunnel Photo No. 11

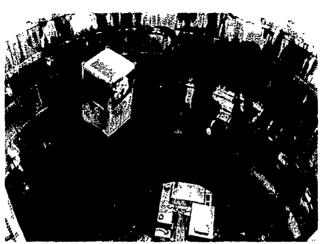


View northeast in San Pedro Creek Outlet Shaft showing soft, massive, unweathered clay shale between elevation 598 and 594, 41- to 45-foot depths.

16 Mar 88 San Pedro Crk Tunnel Photo No. 12

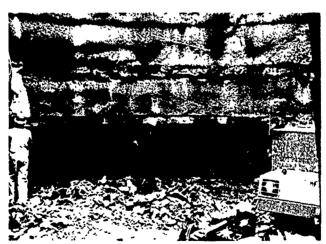
EXHIBIT 6

の東京の情報を表現をあるというと



Man-cage being lowered by crane into San Pedro Creek Outlet Shaft, bottom of 49-foot depth.

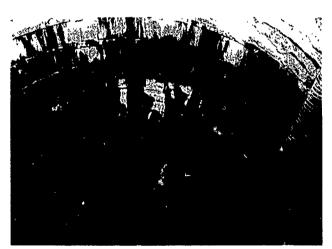
25 Mar 88 San Pedro Crk Tunnel Photo No. 13



Geologist examining soft, massive, unctuous clay shale in the outlet shaft between elevation 594 and 590, 45- to 49foot depths.

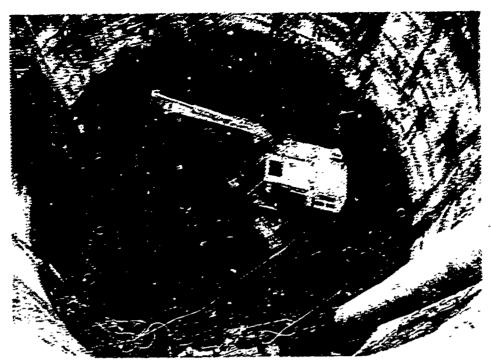
25 Mar 88 San Pedro Crk Tunnel Photo No. 14

Concrete delivered for shotcreting in San Pedro Creek Outlet Shaft.

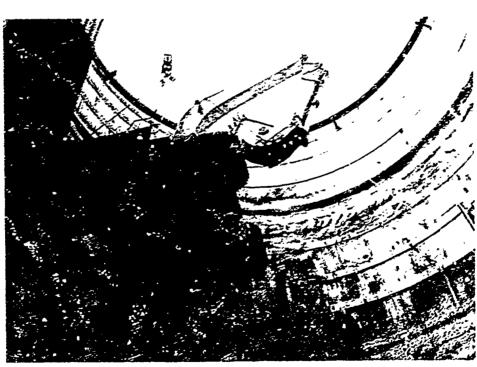


Application of 8 inches of shotcrete over two layers of wire mesh as primary support at elevation 590, 49-foot depth in outlet shaft.

25 Mar 88 San Pedro Crk Tunnel Photo No. 16

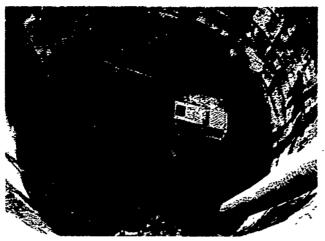


View southwest into cutlet shaft showing backhoe excavation at elevation 585, 54-foot depth.



View of backhoe being hoisted out of outlet shaft from elevation 576, 63-foot depth.

5 Apr 88 San Pedro Crk Tunnel Photo No. 18



View southwest into cutlet shaft showing backhoe excavation at elevation 585, 54-foot depth.



View of backhoe being hoisted out of outlet shaft from elevation 576, 63-foot Jepth.

5 Apr 88 San Padro Crk Tunnel Photo No. 18



View southeast in outlet shaft showing soft, massive, unctuous clay shale between elevation 585 and 581, 54- to 58-foot depths.



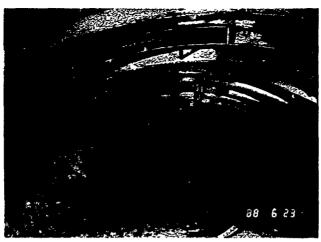
View northeast along southeast wall of outlet shaft showing close-up of the clay shale at the 54- to 58-foot depth.



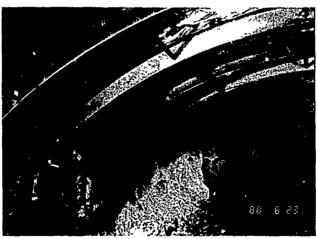
Testing support rock bolts at elevation 557 on east side of San Pedro Creek Outlet Shaft,



Roadheader beginning excavation of horizontal transition from outlet shaft to tunnel at elevation 533 to 522, 106- to 117-foot depth.



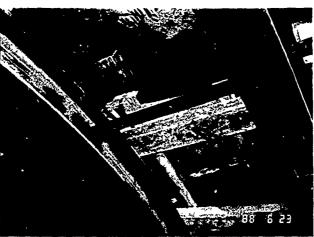
View upstream of transition crown excavation in outlet shaft, showing Ribs E to P. $\,$



Upstream view of crown excavation in outlet shaft transition, showing Ribs D to P. $\,$



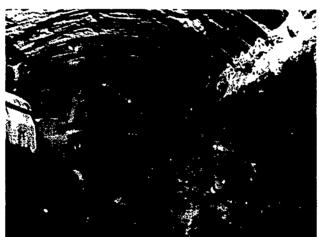
View of west side of crown excavation in outlet shaft transition, showing Ribs E to J.



Cribbing support above Ribs I and J where 6x3x.5' slab of rock fell from crown.



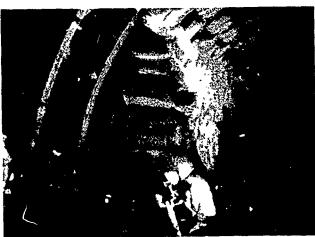
View of steel rib and shotcrete supported crown of outlet shaft transition after vertical shaft excavated another 5 feet to elevation 517.



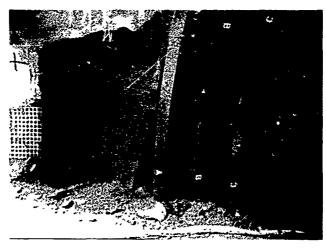
View of backhoe center notch excavation to springline in outlet shaft transition.



Roadheader in San Pedro Creek Outlet Shaft in preparation to excavate for setting transition ribs to springline, elevation 511.6.



Roadheader excavation to springline at Rib F on west side of outlet transition.



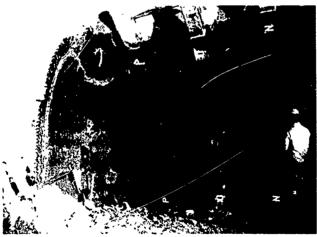
View northwest of outlet shaft transition excavated to springline, elevation $511.6\,$



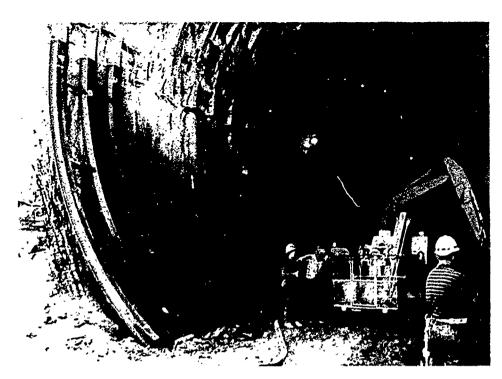
View of roadheader cut to springline at Rib F on west side of outlet transition. Note shallow desiccation fractures in center photo.



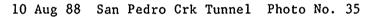
View northeast of outlet shaft transition excavated to springline, showing Ribs K to P. Excavation in soft to moderately soft, massive clay shale.

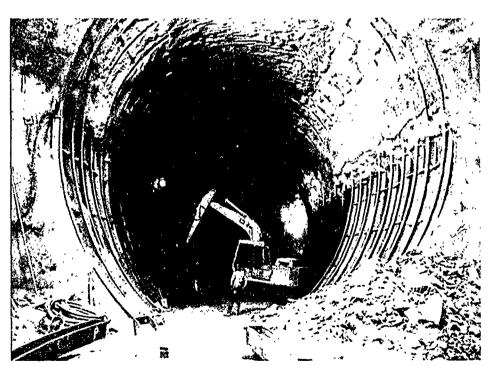


View of tunnel end of outlet shaft transition excavated to springline, elevation 511.6.



View north of outlet shaft transition excavated to invert, elevation 490. Some fracturing in lower 10 feet of excavation.

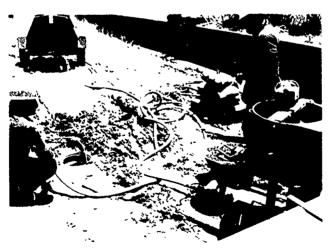




View upstream along fully excavated outlet shaft transition, 60 feet in length.



View southwest of drilling of vent shaft at Station 158+14 near Durango Street.



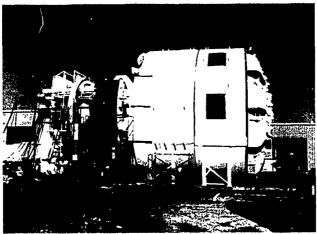
View northeast of grouting of anchor rods for 6-position extensometer at Station $158{\pm}47$ near Durango Street.



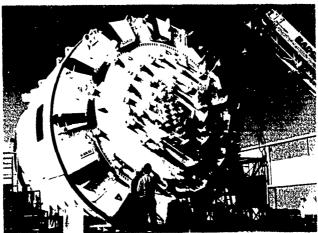
View south of Beck Foundation Company drilling maintenance shaft through upper ring of concrete soldier piers.



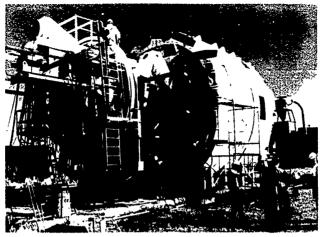
Drilling of maintenance shaft with 6-foot diameter pilot bore and reaming blades to full diameter of 22 feet, 4 inches.



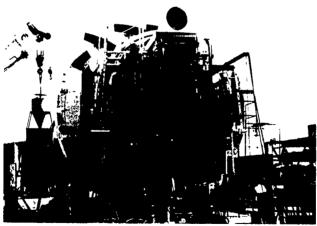
Modified Robbins Model 243-217 tunnel boring machine, TBM, at Boretec, Inc. work yard, 5797 Dietrich Road, San Antonio, TX.



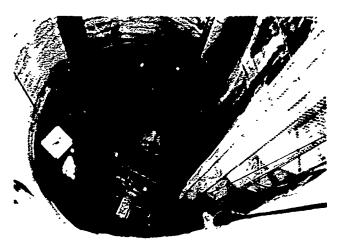
TBM as modified by Boretec from hard rock machine to soft rock machine (26' 11" dia).



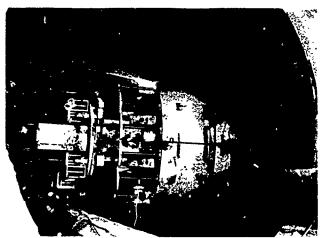
View of back of TBM showing gripper pad on right side and liner erector to the right of the ladder.



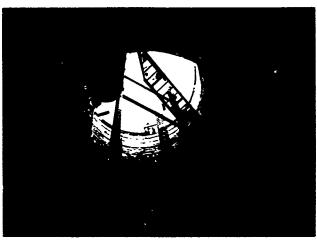
Rearview of TBM being renovated and modified after shipment from the Kerckhoff 2 Tunnel near Fresno, California.



View of TBM being reassembled in San Pedro Creek Outlet Shaft.



TBM parts were transported from Boretec yard and reassembled in outlet shaft.



Installation of supporting frame for muck elevator in San Pedro Creek Outlet Shaft.



View upstream into first curve from about Station 144+50 in San Pedro Creek Tunnel.

Grout batching in outlet shaft for backpack grouting of tunnel liner. $% \left\{ 1,2,\ldots,n\right\}$

1 Mar 89 San Pedro Crk Tunnel Photo No. 49



Batching of neat cement grout at a 1:1 water to cement ratio by volume.



View upstream at grouting jumbo at about tunnel Station 145 + 00.

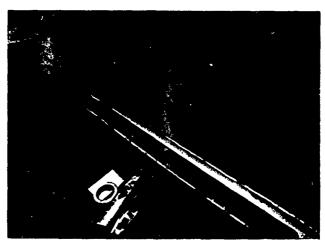
1 Mar 89 San Pedro Crk Tunnel Photo No. 51



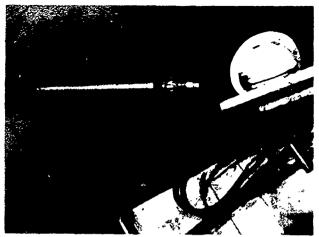
View downstream toward outlet shaft from top of grouting jumbo at about Station $145 \! + \! 00$.



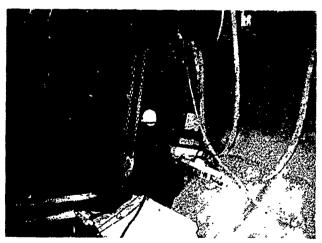
Drilling borescope hole B-5 through TBM tail shield at tunnel Station 143+71.



Core sample of clay shale taken from borescope hole B-6 at tunnel Station 143+71. Borescope holes were drilled to a depth of about 8 feet.



Borescope observation for stress relief fractures in hole B-5 at tunnel Station 143+71.

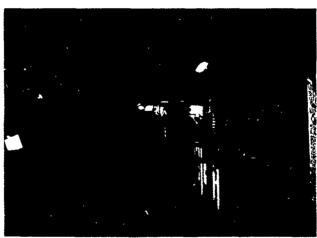


Down-hole video taping borescope holes at tunnel Station 158+39.



View of precast liner segments prepared for erection at back of TBM a^{+} Station 157+38.

16 Mar 89 San Pedro Crk Tunnel Photo No. 57



View downstream along TBM trailing gear. Muck car being loaded in background.

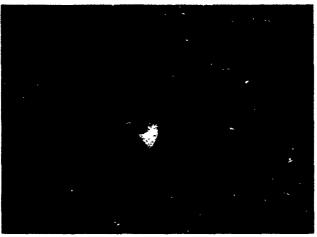


Upward view through TBM trailing gear at Station 158 ± 14 vent shaft intersection in tunnel crown.

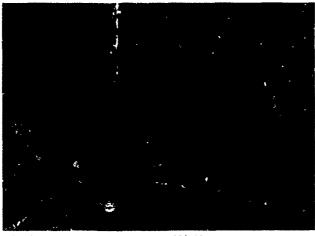
20 Mar 89 San Pedro Crk Tunnel Photo No. 59



Wood lagging and W6x20 steel sets in tunnel crown at Station 158+14 vent shaft intersection.



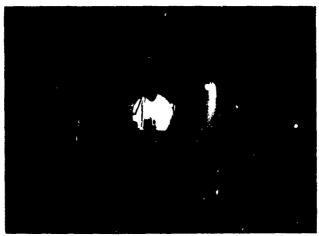
Abandoned water well intersected by TBM at tunnel Station 178+49. Well was partially plugged but produced a steady flow of 2 gpm.



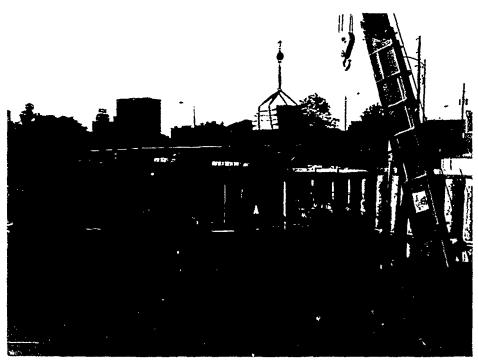
Abandoned water well at Station 178+49 prepared for grouting. 17 Oct 89 San Pedro Crk Tunnel Photo No. 62



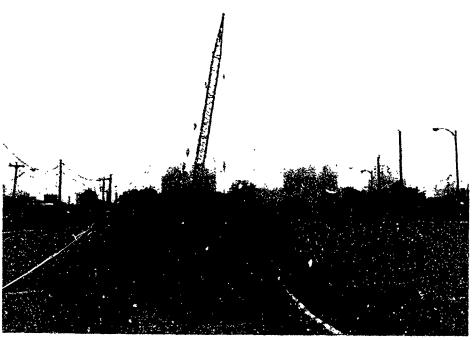
View upstream toward grout batching plant at Station 158+14, vent shaft location.



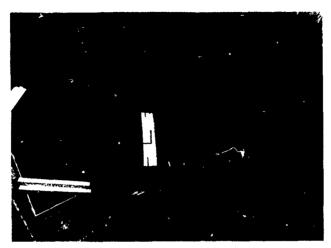
View downstream toward outlet shaft in background. 17 May 89 San Pedro Crk Tunnel Photo No. 64



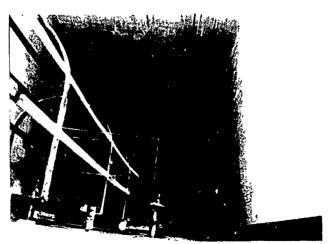
View south at construction of water protection cell to prevent flooding of inlet shaft excavation.



View southeast at San Pedro Creek Inlet Shaft surrounded by water protection cell.



Excavation of upper 21 feet of inlet shaft and reinforcement for rhombus shaped concrete temporary surface structure.



View into inlet shaft from northeast wall. Bottom of excavation at elevation 583, 40-foot depth.



View northwest in inlet shaft showing very limy, moderately hard to hard, clay shale of the M-3 Stratigraphic Marker Bed. The bottom of the concrete is at the top of the M-3 bed, elevation 583.

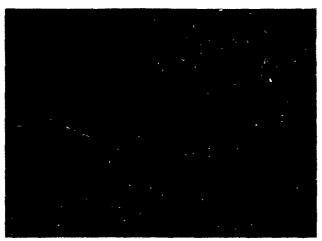
2 Feb 89 San Pedro Crk Tunnel Photo No. 69



View northwest in inlet shaft showing moderately hard to hard, massive, very limy clay shale between elevations 556 and 552, 67- to 71-foot depth.

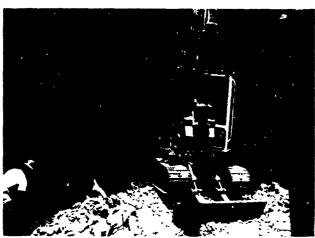
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View southwest in inlet shaft excavated to elevation 536, θ 7-foot depth. Upper rock is very limy and harder than the less limy rock in the lower photo.

31 Mar 89 San Pedro Crk Tunnel Photo No. 71



View southeast toward inlet shaft undercut for tunnel intersection. Shaft excavated to elevation 527, 96-foot depth.

Man de Monar and Marian



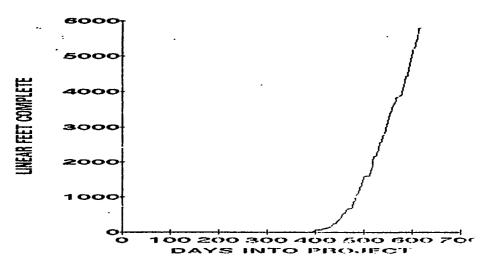
TBM hole-through in San Pedro Creek Inlet Shaft.

APPENDIX B

Tunneling Progress Charts

SAN PEDRO CREEK TUNNEL

PROGRESS CHART AS AT 17 JULY 69



Notes: 1. Day 490 was 7 Har 89 - Scheduled Completion Date.

2. The San Pedro Creek Tunnel is 5843 feet long.

3. Hole through was at 1805 hours on 13 Jul 89.

4. Tunnel excavation was completed on 17 Jul 89.

MACHINE DATA

PROGRESS

Manufactured By: ROBBINS. Model: 243-217/Modified.

Average: 30 ft per working day. Target: 60 ft per working day.

Weight: 550 tons.
Length: 38 feet.

Thrust: 2,640,000 lbs.
Cutters: 57 Discs, 2 Bi-Discs in Centre. Pick option.

Rotation By: Ten 200 HP, 460 V AC Motors.

Guidance: Laser Beam.

Waste Disposal: Trailing Conveyor & Train.

CONTRACT DATA

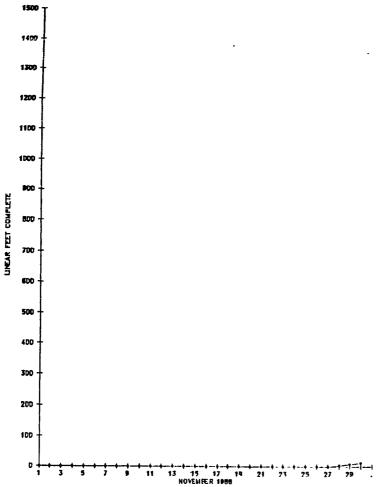
Contractor : OHBAYASHI CORPORATION South San Francisco

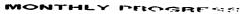
Contract No : DACW63-87-C-0109

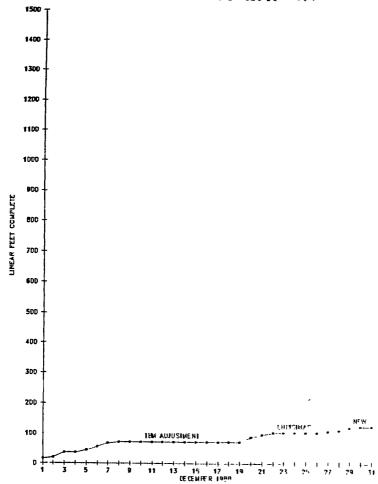
Bid : \$47,750,000.40

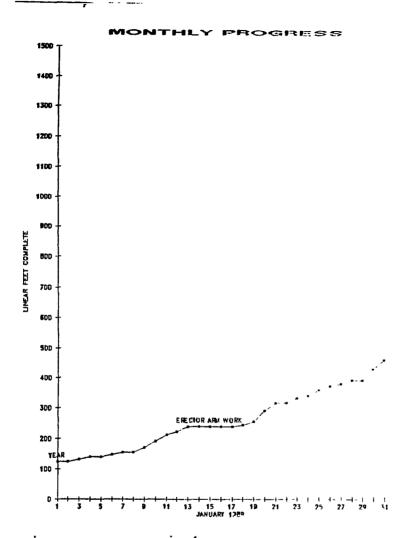
Acknowledged: 3 Nov 87





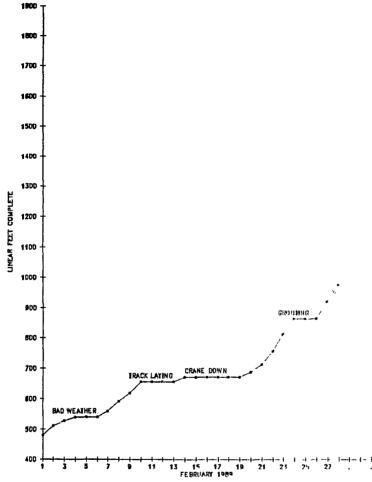






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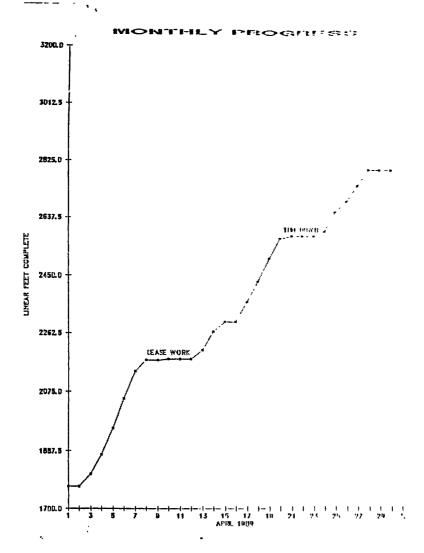


2312.5 - 2125.0 - 212

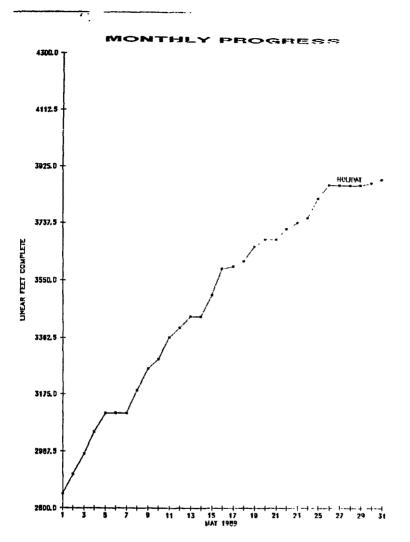
1375.D

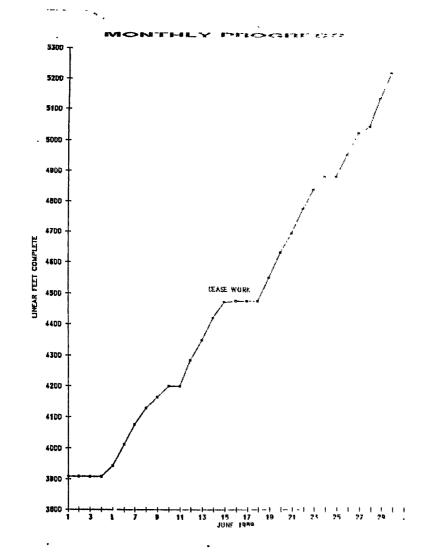
1187.5

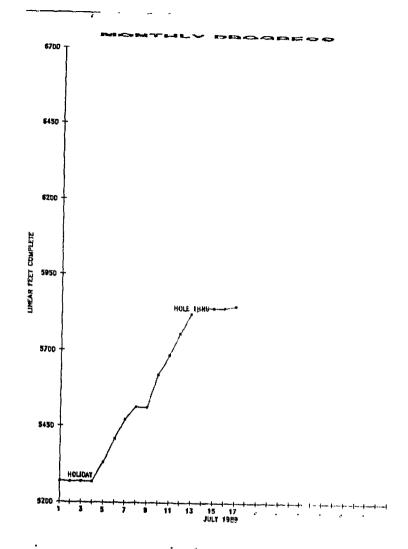
15 17 MARCH 1989



A CONTRACTOR CONTRACTOR







APPENDIX C

Tunnel Liner Grouting Data

. 4			ea gravel a	ND GROUT COX	summion - s	in' funnel.			
	خ ^{ا و} ځ		1	Pea Gravel		Grant			
Veek	End	Rings Placed	Delivered tons	Delivered ou ft	Per Ring cu ft/ring	w jveréd bagszen f1	ler kind on flaring		
Dec	-4	. 9	0	11	£1	U	41		
Dec	11	. 9	109	2291	257	o			
Dec	18	0	O	v		o			
Dec		: 8	39	821	103	O	67		
Jan		<u>.</u> 5	ð	υ	0	υ	"		
Jan		.i . 8	42	881	111	0	"		
Jan		~ 21	64	1347	61	0	O.		
	22	19	63	1326	70	O			
Jan		. 19.	42	881	47	()	U		
	5	37	81	1705	16	O	()		
	12	29	84	1768	61	300	19		
	19 ي	4	41	863	216	465	116		
Feb	26	48	42	881	18	585	12		
Har		53	201	4231	80	779	15		
	· 12	59	191	4021	68	699	12		
	, 19	67	65	1368	20	966	11		
Mar	26	7	0	. 0	0	3159	191		
Apr	. 2	41	124	2610	61	1.330	106		
Apr	.9	· 101	123	2589	26	1811	1×		
Apr	16	- 31	0	0		1286	138		
Apr		69	131	2758	40	3772	55		
Apr		55	0	0	0	812	15		
Hay		79	138	2905	37	2109	27		
Hay		80	231	4863	61 	3231	10		
Hay		63	215	1526	72	1291	68		
May		46	61	1281	28	1100	21		
Jun	-	10		1505		0	.0		
Jun		73	211	1505	62	3279	15		
Jun		69	130	2737	10 50	1641 2156	21		
Jun	25	102	211	5073	517	2196	21		

TABLE 1 : WEEKLY DATA

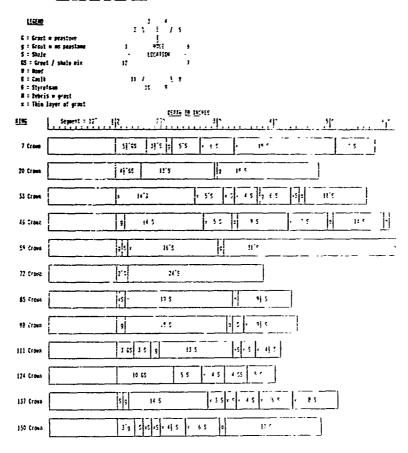
•		Pea G	ravel	Grout			
Month	Rings Placed			Delivered bags=cu ()	Per Ring cu ft/ring		
27 Nov - 1 Jan	31	3115	100	()	0		
2 Jan - 29 Jan	67	1442	66	0	O		
30 Jan - 26 Feb	118	5220	14	1350	11		
27 Feb - 2 Apr	227	12230	54	10233	15		
3 Apr - 30 Apr	256	5317	21	10711	12		
1 May - :28 May	268	13577	51	10737	10		
29 May - 25 Jun	254	12311	18	7066	28		
TOTALS:	1221	56216	16	40097	36 (1)		

TABLE 2 * MONTHLY DATA

NOTE: 1. The average grout figure assumes 1100 ring fully enouted.

INSPECTION OF LINER GROUTING

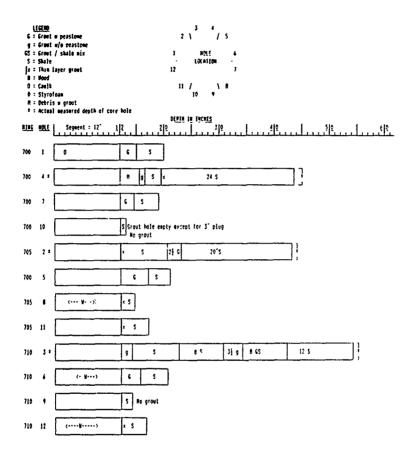
क्का स्वर्क उठा कि स्वरू निर्मा



UER 1 : Grant a peastane g : Gract w 20 peasities 5 : State Hil LOCATION FS : Grast / shale als I . Hoos 11 / 1 = 1 E 9 : Carlt 9 = Styrefsas # = Detris = greet x : This layer of growt tern in boses 1186 略[| Separat : 17' 159 ç 5 2352 3375 -159 4 1 5 8.2 16]"5 259 7 Ç 5 ٤ 159 10 5 5 ç 5 SS 5] 5 3 GS 12 5 364 2 * 144 5 Ç 5 161 1s ç :65 22 5 169 3 : 6 | 5 169 169 9 Grout & loose peastone in grout hole 169 12 <- pipe w jule 5

([210] 6 : Grapt w peastone # : Grest w ac peastone 871E 5 : Shale S : Graet / stale min LOCATION # = Nood C = Caelt 0 = Styrefoan n / ,\ = 19 # : Debris » grout x = This layer of grout PS = Feastone / shale mix GEFIR IR INCHES RISS MOLE | Segment : 12" 174 1 Ç 5 30.2 174 4 2 0 5 174 6 5 s 174 10 Broken badiy 9 5 :FS 17 S 179 2 G 179 5 5 179 8 179 11 ç 10"5 65 81 5 7 5 184 184 6 5 3 65 184 BADLY BROKEN 19 5 • 104 12

3 4 / 5 LEGESS G : Grout a peastone g : Grout u/e peastone 65 = Grout / shale mix POL! S : Shale FOCULTANA? |x : Thin layer grout 12 # : Wood ,\ 1 0 : Caulk 11 / 0 = Styrofoas H : Debris w grost * : Actual measured depth of core hole DEFIR IN INCHES MING MOLE | Segment : 12" ,1/2, , / 2 2 685 1 4 \$ 25 S \$\$ 14,02 23 S z 14.2 685 1 2 **G** 5 685 1 G S 685 10 s 2165 18"5 690 2 1 g 690 Ç 5 g 5 690 (---- ¥ -) . 690 11 5 Wo grout ¢ 692 2 5 495 ÇŞ 6] GS 2 0 10 S 495 ¢ 5 695 Grout hole empty No peastone or grout 695 12 c 5



LEGENO G = Grout # peastone 9 = Greet u/o peastone 65 = Greet / shale ear NOLE S : Shale LOCATION Ix : Thin layer grout 12 W : Nood 11 / ,\ 1 0 : Cault 0 : Styrofean H : Debris w grout * : Actual measured depth of core hele DEPIN IN INCHES #INC MOLE | Segment : 17" 1101 [(---#---**0**-) ¢ 1101 4 ¢ 1181 7 G 1181 10 Core full apart Evidence of grout & 2 shale 1186 2 ¢ \$ 1186 5 ç 1186 8 (-0----y------) Segment only Peastone in grout hole 1186 11 Incomplete core (10° only) 1191 3 ¢ 1191 6 (---¥--) ç 1191 9 No grout 1191 12

16	CEND	3 4
	Grawt w po	
	Growt w/o	
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	Shale	- EGCATION -
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• •	MC1001 DC	
RUBE	NOTE	Openia 19 19 19 19 19 19 19 1
1196	1	(······ֈ·····)
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1201	2	c
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1201	s [Core included segrent only Grout in grout hole
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1206	, [Seguent only to growt
1204	12	

APPENDIX D

Drilled Shaft Logs

1,984.4	MG LOC	P	1510H	western-COE	WSTALL!	1101	5	an	Anto	on10	OF 3 SHEETS	١
PROJECT				San Antonio	Tunne	ls wo	Re	510	ent.	Office See Remains	rk 2	١
San Pe	dro C	reek	Tuni	nel Texas	MSL							١
Statio	on 143	+00 '	(nea	r OutletShaft)	TE MANU	FAC	TUR!		DESIGN O 4 E	ATION OF BAIL		Ì
Beck I	Pounda	tions	5		NOT	L HC	. 01	. 7 'V	743	(45 ton) None	- Turnerimeen	1
			A maio	Hydraulic Instrumentation Shaft		EN 5			74464	None	_!None _	١
Al Mai					15 ELEV	ATIC	M C	ROW	NO WAT	618.2	E1.	١
PHECTIO	07 HOLE				N DATE		_	_	127.45	70	25 20-41 00	İ
20 ven 710				DEF PROM VERT	ME DATE HOLE 25 April 88 125 April 8 17 ELEVATION TOP OF HOLE 638.2 E1.							1
THICKNES					10 TOT	L C	ORE	REC	OVERY	FOR BORING N	/A	١,
DEPTH DE				.2 ft.						n		1
EVATION				CLASSIFICATION OF MATERIA	i Robei				01.	(Dellant time	MARIES	
				(Doacs lpt lord) å			_	Vn	it	weathering o	ater fore deprised in it stantificand	_
638.2	0.0			1-2.01		7	个	1		1. Water		- 1
1	Ε			ay Fill: dark brown				J.	cent	free water	encountered	
	ᅼ		tic	black, medium plas- ity, with scattered		1 1	1	1-	-1	from 20.0 27.0 ft. d		
	. 4		San	d and angular grave	l.	l l	П	1		21.0 It. a	eptn.	
	ーゴ		١	N-0 O'			}	1	1	2 Drilli	ng Method: full flight	
	=		달음	'-8.0' lay' brown to grayis	h	П		1	ļ	A 24" dia.	full flight	
	F_		f Dro	own, mealum prastici	ty,	П	М	1	1	reamed sha	d 27 0 ft. ft to 26" dia	
	ΙŦ		sil	ity in places and		1		1	Ì	set 25 5"	dia. casing	
			Sti	iff.					1	through ov seated cas		
	=		8 0	0'-12.0'			1	1	1	weathered	clay shale.	
	111111111111111111111111111111111111111		C	lay: tan to buff, dium to high plastic	_			1	Qa1			
	l - <u>-</u>		it	y, stiff and damp.		١.	1 0			Continued	boring with Light auger t	_
	1 =		i i			13	casing	1		a depth of	119.2 f. ,	_
	_		12	.0'-20.0'		13	31.5	Ų		cased 119	2 ft bore	
	=			ravelly Clay tar to ayish brown, medium		1.	. 8	-		hole with	schedule 40, ell casing	
	1 =	1	to	high plasticity, wi		1	į G	ł		ie uia.	HELL CESTING	
	1 =			merous calcareous co	n-	1 =	'dia. ca:	-1		Filled she		
	} =	1	an	eations, sand seams d damp		13	5.2	- {		feet of cl	lay fill,	
	1 -	1	1	•		1	Ϊī	١		around 12	lar spaces dia. casing	
	-	7	20	.0'-27.0'	h	1		-		to ground	level and	
	1 =	3	br	layey Gravel grayis	***	1	\prod	-		capped car	sing. Withdre	: W
	1 =	}	l ro	unded gravel with cl	lay	1		1	Qnl	backfille	ing and i with grout	
	1	1	an	d rounded calcareous	; •A	1		J		i		
618.2	20-0	4	\ °°	oncreations, saturate		1		.		Qui-uncon	ric Units.	
	(:	1	27	.0'-39.6'				ļ		alluvial	deposits of	
	-	7	\ <u>\</u>	esthered Clay Shale in and gray, medium	to	1		I			rnary Period.	
	1 :	3	hi	in and gray, medium	•	1				Kt-Tavlar	Shale, clay	
	1 :	3	b1	locky in places, iro	n	1		1	Qal	shale of	the Cretaceou	5
	1 -	3	at	taining along frequen	nt				-GIT	Period.		
1	:	3		ractures and joints, alcareous.		1	1			1		
	-	3	Te	ylor Shale (Kt) of		Į	¥		t	1		
611.2	27.0	1	Cr	retaceous Period.		ŀ	_1_	1		1		
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. Project			San Antonio	Tunn	els Ra	ısıd <i>e</i> r	ntonio nt Office See Rem	or 3 st	ÉE13	
LOCATIO	# (Comp	creek		MSL	14 FOR 21	EVATIO	See Rem	Rich -		
Stati	ion 14	3+00	(near OutletShaft)		IFACTUR	FI DEL	IGNATION OF BAI			
веск	Found	lation	S	Nor	thwest	: 504:	5 (45 ton)			
HOLE HO	14.	-						None	****	
Al Ma	ONILLER		imentation Shaft	14 701	AL MUMBE	n Conf	BOTTS		-	
DINECTIC		É		" ELEVATION GROUND WATER 618.2 E1.						
(30 vente	-	hermej		16. DA1	B HOFE	25	ARTED	160mr Fift	1 00	
THICKNE	SS OF OV	HOUNDE	M'27.0 ft.	15 ELEVATION TOP OF HOLE 638.2 E1.						
DEPTH D	RILLED II	ITO NOC	92.2 ft.	18 701	L CORE	ECOVE	TY FOR BORING	N/A	,	
TOTAL DE	EPTH OF	HOLE	119.2 ft.	is total cone necovery for boning N/A is signature of inspector Robert A, Burns						
LEVATION	DEPTH	LEGEND	CLASSIFICATION OF MATERIA	LS	Casing	Geol.		FMARKS		
	•					Unit	wentlested.	mates lene alve etc. If algettes	(h ~)	
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	1 =		Clay Shale: gray			ĺ	ſ			
	- 글		to light gray, soft to			l	ł .			
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]		zones, calcareous throug	hout		Kt	J			
]	=		but becomes more so with	1))			
1	=		depth, massive, moderate petroleum odor, fossils			l				
i] =		observed in places,			1	1			
			occasional pyrite.			[
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DRILI PROJECT	LING LO	G S	outhwestern-COE San Antonio	Tunne	ls Res	<u>iden</u>	t Office or 3 since is			
San P	edro	Creek	Tunnel Towns	THE BATE	MID TYPE STOR ECE	OF BIT	See Remark 2			
Stati	on 14	3+00	(near Outlet Shaft)	MSL						
Beck	AGENCY	264	Carrecondic)	Nort	hwest	5045	(45 ton)			
NOLE NO	roung	ation	s • m. Hydraulic Instr	19 101	L HC. OF	YFR.	MATURATE UMMATURATE			
		,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	mentation Shaft							
FM LA			•	14 TOTAL NUMBER CORE BOXES N/A 15 ELEVATION GROUND WATER 618.2 E1.						
DIRECTIO	0 P HOL			Jaranten Jeour. Firm						
G vente	·* 🗀	heLm#	DES FROM VERT	1		25	April 88 25 April 88			
			'27.0 ft.		ATION TO					
DEPTH DE				19 SIGN	TUNE OF	INSPECT	OR SOUTHS N/A			
TOTAL DE	PTH OF	HOLE	119.2 ft.		ert A.					
LEVATION	DEPTH	LEGEND	CLASSIFICATION OF MATERIA	\L\$		Geol. Unit	MEMARY COLOR STORE OF STORE OF WARRESTING, are it algorithmen			
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PROJECT San Ped				10 5122	AND Y	41.6	OF BIT	Sec Remark	2	
LOCATION	(Coordina	9 07 310	(near Durango St.)	MSL						
DAILLING	eck Fo	unda	tion	Nort	Wes	t	5045	MATion of Bailt (45 ton)		
KOLE NO	140 000000		ventilation shaf		L HO	OF.	OVFR ES TAKE	None	None	
I Mann	DRILLER		Vencilation shall	14 TOTAL HUMBER CORF BOXES N/A						
. DIRECTIO				IS ELE	ATIO	GRI	OJHD RĀ	fer 623.3 E	1.	
Øvent:			DES FROM VERT	IS ELEVATION CHOUND RATER 623.3 D1. NOTE HOLE 11 May 89 13 May 88						
THICKNES				19 707	ATIO	70	- 05 110	639.3 1	1	
TOTAL DE			98.0	iệ SIĞH	ATURE	OF	HSPECT	OF TOR ROUNCE	22	
	DEPTH		CLASSIFICATION OF MATERI (Description)		_	ng	field Geol. Unit	Chilling time will weatherling at	it of disease	
539.3	0.0		0.0'-1.5'		 	7			·	
259.5			Sand Fill tan, fine	to			~, 1	free water e		
	1.5	_	coarse grained, contain	is some	П	\parallel			3 0 ft dept	
,	∃		fine to med. gravel, or glass and metal debris		П			2 Drilling	Met had	
	크		loose.]]			2 Drilling A 78" din	ull flight	
	=		1 2'-12 0'			11		auger bored	30 ft , set easing throup	
	3		Gravelly Clay buff to				9.1	overburden,	esing throup seited casin	
	=		dark brown, medium to b plasticity, contains so	nigh Pattoro				within wentl	ered clay	
	_=	İ	small to medium rounded	1	[11		shale to ser water Cent:	nued shaft	
			limestone and chert gra soft to stiff	avel,		1		water Cent: clubing with	n 48" dia	
	=		2010 (0 20111					full Clight	nuger to	
	ΙĦ					-		121ft depth shaft to 12	dia ,	
07.0	٦, ا		10.01.02.01		H			backfilled v	vith clay to	
27.3	12,01		12.0'-23 0' Clayey Gravel tan and	d grav				\$	•	
	=		subrounded to rounded					rat ha" dia cacina, bac	pormanent bolling	
	-3		limestone and chert gr		casıng	į	ດາ1	annulu wit	h concrete	
]]		moderately clayey, satuated below 16 0'.	u: =	ES.	R	Yeu			
623.3	16.0				dia	casirg		3 Grologie Qui -Uncons	Units	
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16.3	E		23 0'-39.8'		11	ļ	ļ.,			
10.3	23 0		Weathered Clay Shale:			-	1			
			yellowish tan and gray			Ì	Ì			
	=		medium to high plastic soft, blocky with freq		11					
] =		joints and fractures.	some		j				
	=		iron staining, damp, c eous. Taylor Shale of	alcar-	11			[
			Cretaceous Period.		11		}]		
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PROLL PROMICT an Ped	ING LOG		Outhwestern-COE	Time:	IS For	e den	office See sender Supplied Sed	ar meers		
an Ped	to Cre	ek T	unnels San Antonio. Texas	145.1						
tation	158+1	4.13	(near Durango St.)	W	-	5 66 56 S2 67 5	inimi di finiti Les 100 1			
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599.3	10.02	-						!		
	3		27.8'-121.0' Clay Smale: army, soft	••]			
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l	E		a slight petroleum odor			۱	!			
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COCKE	1584	7 7 7 7	(near Durango St.)	MSL 12				·
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LOCATION	-	THE REAL PROPERTY.		xas	MSL					_		
	eck I	14.13	(near Dura	ingo St.)	North	Jietuni West	## 8693 5045 (45 ton)				
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As bale	7		·		12 ELEV	ATION CA	OW#D ##	FR 673.3	El.	_		
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1. TIOCOVES	S OF OVE	30000					P OF HOL	¢ 639.3	<u>Fl</u>	-		
a. BEPTH CO			98.0		10 TOT /	THE OF	MSPECT	FOR BORRES	N/A	- }		
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ELEVATION	DEPTH:	LEGEMO	CCAMPIC CO	TION OF MATERIA	4.5	Casing	Unit	(Petting thee, a	uates leve des la el Le 11 aspositement			
- <u>-</u> -	120.0	- -	120.0': obs	erved occ. 1	ossils	F123	Kt.		<u>.</u>	- =		
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From Sh Antonio Tunnels

DRIL L PROJECT	LING LO		Vision Southwestern-COE	Tunné	Ation Sa	n Anto: uent O	relea or 4 seces		
San Ped	ro Cre	ek Turr	nel	H SHIE	AND TYPE	COP MT	See Rest th 2		
			Cameron St.)	MSL					
. OMLLAND	AGENCY			IZ MAII	brace "	N'S 019	GRAFION OF BAILL		
Beck Po	undeti.	ODS		12. TOT	MICST 5	OALE FT CEN	Distuncto Distriction		
			Maintenance '	Borthmest 5055 (5 Ton) 13. Total mo or own. 14. Total mo or own. 15. Total mo or own. 16. Total momen cours between None 16. Total mumber cours more None 16. Total mumber cours more None 16. Total mumber cours more None 16. Total mumber cours more None 16. Total mumber cours more None 16. Total mumber cours more None 16. Total mumber cours more None 16. Total mumber cours more None 16. Total mumber cours more					
NAME OF				14 707	AL NUMBE VATION GO	A CORF 1	oars licate		
Al Mann	M OF HOL	.E							
V ERTE	- D	WCLMER			R HOLE	9	May 88 11 August 88		
THICKNES	S OF OVE	RBURDE	16 ft.		VATION TO		LE 642.5		
DEPTH OF				10 107	AL CORE P	ECOVER	V FOR BORING 11/A		
TOTAL DE	PTH 07	HOLE	122.0 ft.	R.A.	ATURE OF				
LEVATION			CLASSIFICATION OF MATERIA		Casing		mitchfiold REMARKS		
			(Peocription)			Unit	to the state of th		
642.5	0.0					 			
ر	! "·"=		0.0' to 0.2' Asphalt parking lot		1	ł	1. Water Level: No free water observed while		
			surface.			l	drilling soldier piers.		
	=				l	,	Miler piers were install		
	=		0.2' to 7.3'			١.	a trace of wetness		
			Clay Fill: brown,	:	1	١,	developed between two		
	=		medium plasticity, mod. sandy to pravelly with	SLIII	1	'	of the piers on the SW side of the shaft.		
į			angular to rounded grain	ns,	ì	1	ator of the shart.		
	=		contains brick and meta	ı	'	1	2. Excavation Procedure		
	=	<u> </u>	debris, dry.		j .	 	Initially a ring of 27		
	-]		7.3' to 16.0'			1	drilled, concrete soldie		
	=		Clay: tan to buff with	n			piers were constructed to just within the top		
	آ۔ ا		some gray mottling, med:		1	i	of unweathered shale.		
			to high plasticity, fine	to:	ì		to depths from 36' to 42 The piers were 36" dia.		
	Ι Ξ		coarse gravel in places. 1" to 2" dia. limestone	, also	١ ا	١.,	The piers were 36" dia.		
	-		1" to 2" dia. limestone concretions in the upper		piers	, 11	and formed an inside		
	_ =		numerous scattered lime	1000	Ĭ	1	shaft dia. of 21'6". The interior of the ring		
	=		pockets, damp below 12'			1	was excavated with a		
	, =		depth.		🛱		backhoe and crane w/skip		
	=		16 01 +0 22 01		soldier		box. This method of		
626.5	16-0-	 -	16 0' to 33 8' Weathered Clay Shale		ه ا	 	excavation continued		
	=		tan and gray, high plass	ticity.	concrete	Кt	below the pier bottoms to the 50' depth, with 6" of shotcrete support		
	=		compact, soft, blocky,		2	J	6" of shotcrete support		
	=		occaional fossils. Taylor Shale (Kt) of the		8		helow the piers. The remainder of the shaft		
	=		Cretaceous Period.	-	, n	1	was diilled and reamed		
	=				dfs	1	was drilled and reamed to a dia of 22'4" and		
	7	'	33.8' to 42.5'		125	l	to a total depth of		
] -	i	Clay Shale	_	٠,	1	122.0 ft.		
	=		Unweathered except along fractures and joints,	3	1	l	Sodier piers ware		
	=		dark gray with reddish		l	1	Sodier piers were constructed by Cato		
			dark gray with reddish tan 1" to 2" oxidized		1	1	Electric and Drilling.		
	=	ì	seams along fractures		ì	1			
	ιΞ		and joints, massive,		1				
	-	l	soft, cont ains nearly horizontal joints at 40	,	1				
	=	1	and 42' depths with		1	1			
	=	1	several high angle		1				
	=	1	Joints and irregular		ì				
	=	l	fractures.		1	Kt	[
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S. HAME OF	MILLER			14 TOTAL NUMBER CORF HOXES HORE							
Al Mann	OF HOL		·	IS ELEV	ATION G		En		1		
E. DIRECTION			DES FROM VERT	16 DATE	HOLE	17.7	Any PB 11	August 88	1		
				IT ELEV	ATION TO	P OF HOL	6h2.5		1		
7 THICKNES							ron noming N	/A	1		
9 TOTAL DE				1		INSPECT			1		
			122.0 ft.	LR.A.			ntchfield nrma	nrs	١.		
ELEVATION	DEPTH	LEGEND	1-0001 01100		Casing	Unit	Peliting time, wat	or ices derited	Į.		
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	=	İ	2.5 to 122.0:			1 1	Qal-Unconsol	idated	Ι.		
	_	l	Clay Shale: light gray to dark gray, soft to		l	1 1	alluvial dep		[
1	=	1	to dark gray, soft to			1 1	Quaternary P	erioa.	1		
1	\	}	moderately hard in limy zones, massive, breaks		1		Kt-Taylor Sh	ale,	1		
	=		predominantly with conch	oidal	1	Kt	clay shale o				
1	_=	1	[fracture, calcareous to	verv	l		Period.		Į.		
	=	}	calcareous or limy, occa fossils, scattered pyrit	sional	l				:		
ļ	ΙΞ	1	crystals, petroleum odor	•	1	1			1-		
1	60.00	1	Taylor Shale (Kt) of		1	1			1		
1	=	1	Cretaceous Period.		1	1			1-		
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ŀ	1	7	petroleum odor.		²	1					
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	ro-Cree		•	Tunnels Rections Office or & success to size and tyre or net See Regul & ? IT BATHER FOR ECEPATION SHOPE (1994 2 HST)						
Station	181+7	.08 (C	el amaron St.)] Hill		******	611667FR4			
· DIMILLINIE	AGENCY			_North	west_50	45_(45		Hone		
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. NAME OF E			(Sixti s	. 14 1011	L NUMBER	COME I	OXIS MORE			
A1 Henn	0 F HOL	ŧ	·	·		7277	# * * # * * * * * * * * * * * * * * * *	654FL F1F5		
(X) ventre				IS DATE			May En	11 August 88		
THICKNES				10 TOTA	ATION TO	COVER	r for Roung	u/A		
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PROJECT San Pedi LOCATION	o Cree	k Tunn	el	13 317 E	Tunnels Resident Office OF 1 Meris 13 SHE AND THE OF MY See Remark 2 11 SATURYON ELEVATION SHOWN 15M - MEL							
Station	181+77	.08 (c	emaroń St.)	1431. 12. MAR	IFACTURE	TTEEN	MATION OF DRILL					
OMILLMO.	ASENCY			_Nort	hvest 5	045_(45	Ton)					
HOLE NO		en chamb	Maintenance Shall	BUR	AL NUMBE	SHAF ES.	None None					
HAME OF E					VATION GR	OUND WA	TEN					
Al Mann DIRECTION (V) VERTIC				10 DAT	E HOLE	19	May 88 11 August	88				
			16 ft.	17 ELE	VATION 10	P OF HO	.c 642.5					
DEPTH DA				19 101	AL CORT P	HSPECT	FOR BORNING 11/A					
TOTAL DE	PTH OF	IOLE	122.0 ft.	R.A.	Burns I	Roy C	rutchCield					
LEVATION	DEPTH	FEGEND	CLASSIFICATION OF MA	TERIALS	Casing	Geol. Unit	REMAIRS [Pelling time mate type desired westering at it significat	h^1				
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DRILL	LING LO	x Sc	outhwestern-COE	Tunne	els R	San A	t Office or 3 surers	ı				
i, Phosect			San Antonio	10. 347 €	AND TYP	F OF P17	See Remark 3					
				M. SIZE AND TYPY OF MY SEE REMARK 3 IT SATURYON ELEVATION WISH HISH MEST								
Camaro	n & 1	Salina	s St.,Sta.185+73.90	15. MYM!	IF ACTUR	ienis de u	HATIFM OF BRIET "" " "	ł				
λ. H.	Beck	Pound	lation	IT HAUFACTURER'S OF WIFFALTON OF DRILL " NORTHWEST 5045 (45 ton) STATEL NO OF OVER INSTRUMENTO MARKETS TAREN NONE NONE								
HOLE HO	(As steen		led Mile	IS TOTA	IL NO O	TOVER LESTARE	None None					
NAME OF			ventilation shad	1 TALL			mers N/A					
Al Mar				is FLÉS	ATION	MUNIT #4	*** 630.0 El.					
L DIRECTIO				M DATE		- 17.0		ì				
ED ventu		**************************************	DEG FROW VERT			_ 2	May 88 ,4 May 88	•				
THICKHES	S OF OVE	HOURDE	• 13.58 ft.	7 (15)	ATION 1	OP OF HA	643.0_E1.	l				
	ILLED H	TO RUCK	103.42 ft.	19 300	AL COMP ATUME O	FIÑSLIČI	on NA	l				
TOTAL DE	PTH OF	HOLE	117.00 ft.	Roy	Crut	prield,		i				
ELEVATION	DEPTH	LEGEND	CLASSIFICATION OF MATERIA (Description)	LS		; (col.	REMARKS Fritting time uning t == 1 cet of under the set of the se	1				
•		•				linit		۱.				
643.0	0.0□		0 0'-2 5'		111		1 Motor Lovels	1:				
	=			ray,		1 "	1 Water Level. Some free water was	1				
		 	low to medium plasticit	y, ,		-	observed flowing from	1:				
	Ξ	1	gravelly with chert nod	ules,		l	red oxidized, jointed	1				
	_	1	some brick, ceramics an	ď		Qal	red oxidized, jointed clay at 13.0' to 13.58'	١.				
	=	1	other debris scattered throughout				depth	1:				
	=		_			1	2 Clas Drate Wile	L				
	=		2.5'-5.0'			1	Daraged 6" dia. drain	1				
	=	i	Clay: dark gray,			1	? Clay Drain Tile Daraged 6" dia. drain tile at 2 ft. depth	1:				
		1	low plasticity, soft, t	hin				1				
	=	1	soft caliche layer near basal contact.			1		ĺ				
	=	1	near ousax contact.		1 = 1	1		١.				
	_	1	5.0'-13.58'		8 1	1		1:				
	_	}	Clay tan and gray mot high plasticity, soft,	tled,	"	ŀ		١.				
	-	ł	high plasticity, soft,	blocky	اقا	1		1:				
	_	1	moist, with red iron st	ains	₽	1		-				
c 1-	13.58	1	moist, with red iron sta along healed joints, co small scattered limy po-	ntains	<u>ا</u> يوا		3 <u>Drilling Method</u> . Augered a 78" dia. borin	ľ				
629.42	13.56	1	small scattered limy po-	ckets.	١٣١	Į.	Aupered a 78" dia. borin	4				
	=	1	13.58'-31.0'				to 17 ft., reamed to 96" dia. and set 96"	ı				
	=	1	Weathered Clay Shale				dia. casing	1				
	l - <u>-</u>	j	Yellowish tan and gray		111	1	Continued with a 78"	1				
626.0	17.0	1	mottled, soft, blocky,		Y		dia full flight auger	ŀ				
		}	highly jointed and frac		1 1	kt	to the 117 ft depth	ł:				
	ļ =	3	with red iron staining a upper contact.	atong	! !	1	Rackfilledto 114ft ,	1:				
	1 =	7	Taylor Shale (Kt) of]	ft , filled the re-	ŀ				
623.0	20.0	7	Cretaceous Period.		1 1	1	maining annular space	1:				
	۱ -	7	1		1 1	1	with concrete, pulled	1:				
	=	7	ŀ		1 1	ĺ	96" dia. casing,	ŀ				
	1 =	3			l i		sealed off shaft opening	3				
	=	1				ŀ	with metal plate (refer to remarks 5%6 to explain	٠Į،				
	1 -	1	1		1	1	thicker concrete shaft	Ť				
	1 =	1			"		wall)	1				
	=	1	[casing	1	{	1				
	-	1	1		88		Ī	1				
	=	1	1			1	Ī	1				
	1 .	Ė			dia.	1	Į.	1				
	1 =	3	01 217 01		1 -	1	1	1				
	-	3	31.0'-117.0' Clay Shale:		φ. ω	1	1	1				
	1 -	- i	lightgray to dark gray,		1 1	1	1	1				
612.0	31.0	-	soft to moderately hard	•		1		-				
		7	massive, has mild to				1	ı				
	-	7	strong petroleum odor,		Į Į	Ţ		ı				
	:	7	contains occasional fos	8118		1		١				
	-	3	ard pyrite crystals, calcareous to very calc	arenue	J			ı				
	1	-	calcareous to very calc becomes moderately hard		1	1		ı				
1) :	4	nn limy zones.		1	Ì		١				
	1 7	7	Taylor Shale (Kt) of					-				
1	:	7	Cretaceous Period.			1		1				
i	-	7	}		1	1		١				
1	1 :	4				١,,		Į				
603.0	40.6	-	1		1 1	Kt		1				
					CHOIL	CT CT	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	. وب				
MAR 71	10 36	PREVI	OUS EDITIONS ARE OBSOLFTF		' Se	n Antor	io Tunnels SP-4					
			(THAN STICENT)									

D-12

	MG LO	_ I'	out	hwestern-COE	Tunne	ls Res	an Ai	office	or 3	2 5HFE15
n Pe	dro C			San Antonio	IN SIZE	MO TYPE	OF RIT	See Rema	rk 3	
CATIO	10	HOO at 1		st.,Sta.185+73.90	MSL		#			
ULLING	AGEI CY Beck	Pour	da+	ion	North	west!	5045	(45 ton)	- 	
	(As alone			prio !	11. TOTA	EN SAMPL	OVFA ES TAKF	(45 ton) None		one
	DRILLER			ventilation sha	1074	. MINIST	CORF R	OUFS N/A		
l Mar	13 H OF HOL						OUND WA	ren 630.0	COMPLETE	,, -
				DES FROM VERT.	M DATE		[2	May 88 .	4 May	88 -
HCKNES	s of dve	RBURG	EW	13.58 ft.		ATION TO		ron noning N		
	HLLED IN		CK	103.42 ft.	TO SIGN	TURE OF	MSPECT	ÓR	<u> </u>	
	PTH OF		т-	117.00 ft.		Crutch Casing		ne.	MARK .	
VATION	DEPTH	LEGE	٣	CLASSIFICATION OF MATERI (Description)		Carozing	Unit	(Petting time (weatering o	eping tran st is it espects B	
3.0	40.0		╅					4. Geologic	Units	
	=		1.	0 01-le 01. 11-la				Qal - Uncon alluvial do	solidate	
01.0	42.0	1	4	2.0'-45.0': Light gray mssive, moderately so	rt,			the Quatern	ary Peri	od.
	1 =		¥	ery calcareous or lim	٠ <u>.</u>]	Kt - Taylor	Shale	
	-		ļ			l		clay shale		ıceou
	=	1				1		Period.		
	-	1	ļ			l	١.,]		
	=	1					Kt			
	1 -	1	1			1	1	}		
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83.0	60.0	7	- 1				1			
	1	3	-			dia.	1	1		
	1	7				189				
	-	Ξ	ļ			-	Kt	1		
		=	- 1				١.			
	1 -	=	- 1	65.0'-70.0': Light gr	AV.	1	1			
578.0	65.	9		massive, moderately s	oft to		1	1		
	-	╡	1	moderately hard, high	ly	1				
	ĺ	Ξ	-	calcareous, with occa fossils, pyrite cryst	als,	l	1			
	-	7	1	and mild petroleum ou	or			1		
	1	3	ļ	when broken.		1	-	1		
	-		- [1				
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	1 24000000						frate fea	
DMLL	ING LO		outhwestern-COE	Tunne	ATION :	san A siden	ntonio sett 3 t Office or 3 sertes	
L PROJECT	dea (San Antonio Tunnel Texas	10, SIZE	AND TYPE	OF \$41	See Remark 3	
2. LOCATION	(Courdin	eres es 5	السائفة	MSL				
Camaro	ASSECT		as St., Sta. 185+73.90	Nort	UFACTURE hwest	#'s besi 5045	(45 ton)	
A. HOLE NO.	Heck (As the	Foun	dation	19. 101	AL NO OF	OVFR ES TARE	(45 ton)	
S. NAME OF	BRILLER		ventilation sha	4 101	AL NUMBER	CORE C	oxfs N/λ	()
Al Man	n			15 ELE	VATION GR	OUND WA	ren 630.0 El.	<u> </u>
6. DIRECTION	-		D DEG FROM VERT	M DATE		_ [2	May 88 _ 4 May 88	
7 THICKNES					VATION 70	P 07 HO	Lr 643.0 El.	
• DEPTH D7				79 5164	ATURE OF	INSPEC	Y FOR DORING N/A	
9 TOTAL DE	PTH OF	HOLE	117.00 ft.		Crutch		REMARY	
ELEVATION	DEPTH	LEGEN	1	IALS	Casing	Geol. Unit	(Pelling time unter lone stepth of meeting ate it nightlis and	
	_ <u></u> _		 			-	·	_
563.0	80.0-	i					}	7
	_	ļ						-
	Ξ	1	1			1		
		1						
558.0	85.0	}	85.0'-117.00': Light	gray,		1	5 85 0'-117 0' Clay	-
	_	1	85.0'-117.00': Light massive, mod. soft to)	1	kt	shale became harder,	<u>-</u> -
	ΙΞ	1	mod. hard, very calcareous, with many		1	l	driller exchanged flat aurer teeth for pointed	=
ļ	=	1	fossils, large pyrite crystals and mild pe	•	1	1	"Tiger Teeth", and pumped water from creek	-
	ΙΞ	1	odor when broken	r.otemi			into shaft to assist	<u> -</u>
55	90.0	1				ļ	sinking operation	<u> -</u>
 "	90.4	3	}		1		6. 90 0'-117.0' Shaft	!:
		}				1	began to plunge towards	-
	=	1			1	1	the west (San Fedro Creek 24" dia and 36" dia	=
1	l <u> </u>	1					pilot holes were bored	[
1	=	1			½		to help re-align shaft and guide 72" dia auger]:
	=	1	ļ		casing	ļ	The hole was then reamed out to the 78" dia	
ł	_	Ε.			i	1	out to the 78" dia	ļ:
1	=	1			dia	1	Alignment rechecked and	<u> </u>
l	=	:			.84	ļ	found to be within contract tolerances.	=
510.0	100.0	1			1 3	1	onortic boat ances i	<u> -</u>
543.0	1 -	-1			1	1		=
	1 3	3			1	ļ]:
	=	7						=
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ļ	1 -	3			1			[=
1	:	#						1
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		∃						-
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1		Ξ.				1		1
529.0	114.							
1		3			Clay	1		1
1	-	=	1		F111			1
526.0	117.	₽	Shaft bottom					-
	-	4			1	ĺ		
1	1	Ξ			1	1	1	1.
L		ユ.			- FOIC		100 7 119	ــــــــــــــــــــــــــــــــــــــ
ENG FOR	IM 18 3	6 PRE	EVIOUS EDITIONS ARE OBSOLFTE		1	an Ant	onio Tunnels SP-4	

Date	LING LO	× 1	Division Court burns to a second	intra.	ATION	San Ar	ntonio meri
PROJECT	THE E	~	Southwestern-COE	Tunn	els R	esider	t Office or 3 surcis
San Pe	dro C	reek	San Antonio, Tunnel Texas	10 47 F	WILL TAL	CEVATION	See Remark 2
statio	76199	F81 -	I (near Inlet Shaft)	MSL			- · · - - /
1 DRILLING	AGENCY		The state of the s	Nort	m ii tun hwest	5045	(45 ton)
A. H.	Beck 1	Found	lation	12 101	AL I'C O	OVER	143 (011) — —
			I (near Inlet Shaft) dation mentation Shaft	}- ^···	m ni tand	LES TAVE	None None
Al Man	DRILLER		Tiblic During	14 101	AL MITME	A CORF	M/A
6. DIRECTIO				-	VATION G	HOUNG P	630.8 El.
*Tventi			D DFG FROM VEHT	l	₹ 1171 €	28	April 88,28 April 88
7 THICKNES	s of ove	ROURD	FM 11.8 ft.	17 51.5	VATION F		" 035.8 El.
0 DEPTH DE				18 701	AL ! THE	RECOVER	TOR BODIE N/A
9 TOTAL DI	PTH OF	HOLE	107.0 ft.	Rob	ert A.	Burns	ron
ELEVATION	DEPTH	LEGEN	CLASSIFICATION OF WATERIA	.,		Genl	IPriling t in agint from storth t
<u> </u>			4		ļ	Unit	most of m, ofc if pig tti and
635.8	0.0'		0.0'-0.83'		1		1. Water Level
i I	=	i	San Pedro Creek Concre	te	. 5		trace of free water
i			San Pedro Creek Concre Liner: 10" thick, rein with #3 bars.	force	1	1 "	observed at 5 ft. depth
]	=		with #3 bars.			1	2. Drilling Method
			0.83'-11.8'			1	Bored through San Pedro
630.8	5.0		Weathered Clay Shale:			1	Creek liner with 54"
030.0	2.0		tan and gray mottled, h	igh		1	dia. core barrel, inserted a 54" dia.
	=		plasticity, soft, somew blocky, moist. Taylor S	hola			casing 3 ft below
((Kt) of Cretaceous Peri	od.		ĺ	top of concrete liner
	_		į.				to divert Creek water around shaft.
	=		11 8'-107.0' Clay Shale.		100		
	_		gray, soft, massive, wi	t.h	casing	ſ	Replaced 54" dia. core
624.0	11.82		1 1t. Bray moderately sof	t.	88		barrel with 36" dia
	=		limy zones increasing w	ith			full flight auger. Bored 20 ft. and set
	-		depth, contains fossils and pyrite crystals, em		dia.		36" dia ensing within
Í I			a strong petroleum odor		يو ا	Kt	unweathered clay shale.
	_		Taylor Shale (Kt) of		m		Continued about at the continued and the continued about the continued about the continued and the con
			Cretaceous Period.				Continued shaft sinking using a 24" dia full
ĺ			1		{	1	flight auger to 107 0 ft
	_				1		flight auger to 107 0 ft depth, cased full length of shaft with 12" dia.
	7					ł	casing, and seated it
i i	\exists		{			1	on backfilled clay from
	=				i	}	105 0' to 107.0'.
615.3	20.01		~		١,	1	Rackfilled annular space
	-1					İ	Packfilled annular space around 12" dia. casing
	-					[with Arout.
	3						Removed 36" dia
]	=				50	1	casing.
{	=				casing		Cot = 2h" at min .
	-1				ಕ	Kt	Set a 24" dia. CMP in grout to stand 2 ft.
1					dia	1	above the creek.
		1	1		_	l	J .
	ㅋ				12"		When grout had hardened, removed 54" dia. casing
	1					1	and capped 12" dia.
1						1	casing.
	\exists						
1 1	-				İ		
1 1			į.			l	
, ,	-1						
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[_				ĺ	1	
	-	i					
							1
]	=					}	
595.8	40.0				,	ļ	
			MIS EDITIONS ARE ORTOLFTE		-1,-,-	L	o Suppole
MAR 71	.0.10	FREVIO	TRANSPUCANTI		' San	Antoni	o Tunnels SP-5

D-15

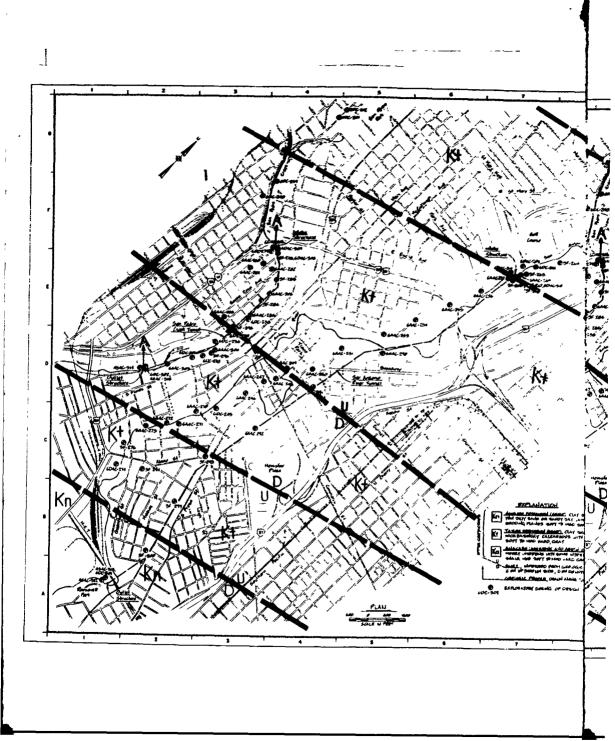
(TRANSLUCENT)

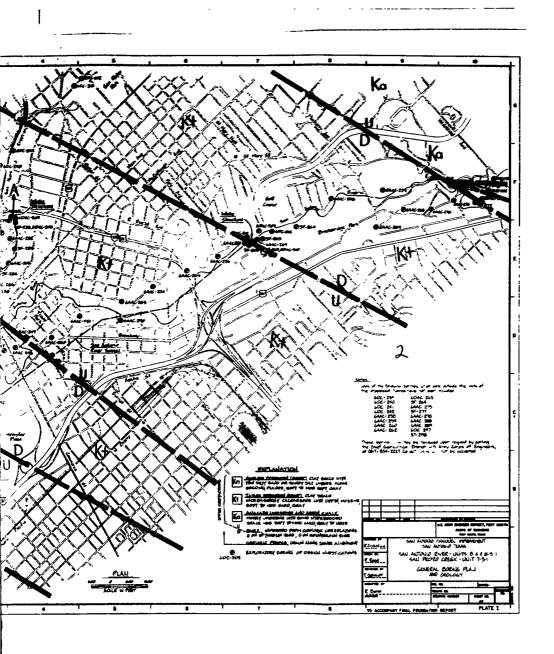
		166	rision ~	1	100 E	-11 AD	onis min
	ING LO	5 5	outhwestern-COE	Tunne	ls Re	ident	
PROJECT	ro Cr	eek 1	San Antonio, unnel Texas	10 5177 11 11211	AND TYPE	OF PIT	See Remark 2
tation	1001	ัยเรีย	unnel Texas (near Inlet Shaft)				
DIGILLING	AGENCY			North	west	3045	(45 ton)
HOLE HO.	eck F	ounda	tion Hydraulic Instr	12 1866	L PO OF	24.17.	
			mentation Shaft	14 1014	L HUHPET	CORF #	OFFS N/A
HAME OF E				IS ELFY	ATION GR	ORNU MY	630.8 El.
DIRECTION			OFG FROM VENT	IS DATE	HUF &	28	April 88 28 April 88
KJVERTIC				IT ELFY	A110N TO	e or not	ε 635.8 El.
DEPTH DR				18 771	I CORF R	rcovra	r con nomine N/A
TOTAL DE			107.0 ft.	1	rt.A. J		ሳብ
LEVATION			CLASSIFICATION OF WATERI	AL C	'a' 111		REMARKS
		•	greatigates;			unit	(Palling t in mater time destitue), manifest of, ate, il all different
595.8	40 04		For general week olers	(ftc=-			3. Geologic Units:
1	=		For general rock class: tionsee sheet 1.	11 1Cg.			Qal-No unconsolidated
	-						alluvial deposits were encountered at this
1	=				1	Kt	site
						l	V4 M1 01 1
{	111111111111111111111111111111111111111	1	ļ		\	1	Kt-Tavlor Shale, clay shale of
			İ				Cretaceour leriod.
	-	'			1	1	
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	<u> </u>	}	1			1	
	_ =	1					
580.8	55.0	1	55.0'-58.0'. Light gray			1	1
	==	}	highly calcareous or 1 moderately soft to mode		-	1	4. Driller replaced
	١Ξ	1	hard, massive, with oc	casion-	•	1	flat tipped Auger teeth with "Tiger" teeth
	-	:	al fossils and pyrite	crystal	. B		to bore through hard
	:	1			casing	1	limy clay shale.
575.8	60 a	Ľ			",	1	
	60.0	:			dia.	1	
1	_	4	1				
1	1 :	-	1		#		
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1	1	-			1	1	
	1 -	=	76.0'-107.0': Light gr	ay,			1
1	1	:	highly calcareous, mas moderately hard, very	foss-		1	
ļ	-	-	moderately hard, very ilferous and abundant	large			
1	1	=	pyrite crystals, mild		1		
559.8	76.7	01	petroleum odor.				
1	1	3			-	1	
1	Ι.	_			ļ		
						1	1
555.8	80.	=			1	1	SP-5

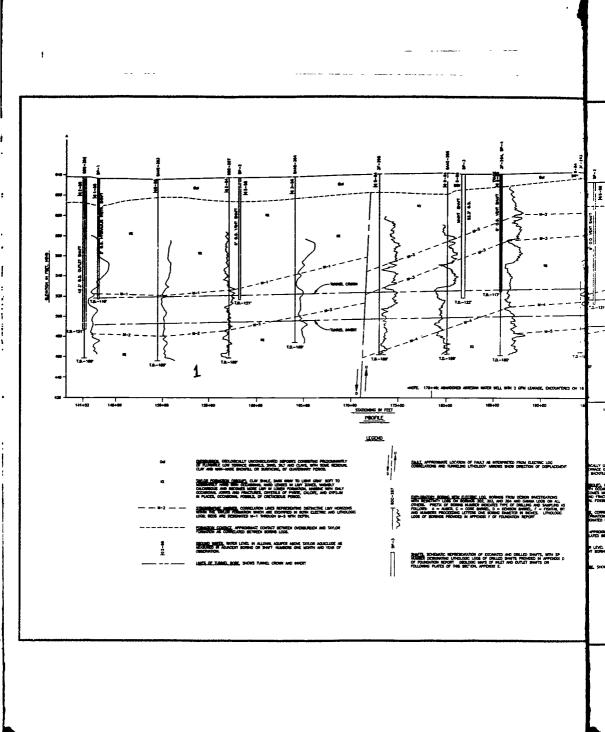
DRILLING LOC SOUTHWESTERN-COE SON ARTONIC Trimon's Recedent Office of 3 series of the property				IVISION		INSTALL	ATIO			-pairi -	٠,	
Station 199931 (near lalet Shaft) Brail and the property of expansions of control of the property of the prop		LING LC			rn-COE	San An	tonic T	innels	Resident Of	or 3 sure	12	
SELICION POPONICA (SECRETARIA) SELICION SPACE (SERVINATION CONTROLLER AND CONTRO		to Cree	k Tun	nel. San A	ntonio. Texas	IN SIZE	AND TYPE	OF PIT	ing-lightly i	93		
The first and the control of the con	Station	199+3	oren er s	r Inlet Sh	aft)							
THE CONTROL OF THE PARTY OF THE						l_ lior	thuest.	5054 (45_Ten)	- 		
A J Meno. - ORIGINAL PROJECT - ORIGINAL PROJ	MOLE NO	14.	mintio	me me Hyd	raulic, o	13. TOTA	L NO OF	CY TAKE	None	None	"	
THICKNESS OF OVERBURDEN 11.8 ft. THICKNESS OF OVERBURDEN 11.8 ft. TOTAL DEFTH OF MOLE OF THE LEGEND CLASSIFICATION OF MAY EARLY DESCRIPTION OF	HAME OF	DRILLER		i ths	trument Shaft					_		
THICKNESS OF OVERSUADEN 11.8 ft. THICKNESS OF OVERSUADEN 11.8 ft. TOTAL DEFTH OF MOLE 107.0 ft. CLASSIFICATION OF MATERIALS CRASSIF ROUND TOTAL DEFTH LEGEND CLASSIFICATION OF MATERIALS CRASSIF ROUND CLASSIFICATION OF MATERIALS CRASSIF ROUND Kt Kt Clay Fill 535.8 100.07	A) Mano	OF HOL	ě					OUND WA	630.8 E1		1	
SERTH ONILLED MTO ROCK 95.0 ft. THE TOTAL CORE RECOVERTY TOR BORNE II/A TOTAL CORE RECOVERTY TOR BORNE II/A TOTAL CORE RECOVERY TORS BORNE II/A TOTAL CORE RECOVERY TORS BORNE II/A TOTAL CORE REC					DEG. PROM VERT			20	April 88 1		3 _	
TOTAL DEPTH OF MOLE 107.0 ft. CLASSIFICATION OF WATERIALS CLASSIFICATION OF WATERIALS CLASSIFICATION OF WATERIALS CRaing Geol. Unit Kt Kt Class R Special A	THICKNES	s of ove	ROURDE	N 11.8 ft	•	17 ELEV	ATION TO	P OF HO	535.8 E1.	17.6	[
Clay Pill Pill Pill				* 95.0 ft		H SIGN	ATURE OF	MSPECI	OR BOHING I	1/A	`	
555.8 80.00 Kt						Robe	rt A.	Furns			1	
535.8 100.00 528.8 107.04 528.8 107.04 528.8 107.04 528.8 107.04 528.8 107.04	LEVATION		LEGEN	CLASS		^LS	Casing	Geol.	Delling time, w	ateries depte	, I	
535.8 100_07 Clay Fill	555 0		┝∸	 				0.110				
535.8 100.00 	,,,,,	00.0	į	1							- 1	
535.8 100_G		=	1	1								
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535.8 100_GT		=	1	1			5					
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28.8 107.04 F111		1 3	-	1			03-11	1	1			
	28.8	107.0	4					1	1			
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milmin		1	1	1			1	1	1			
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APPENDIX E

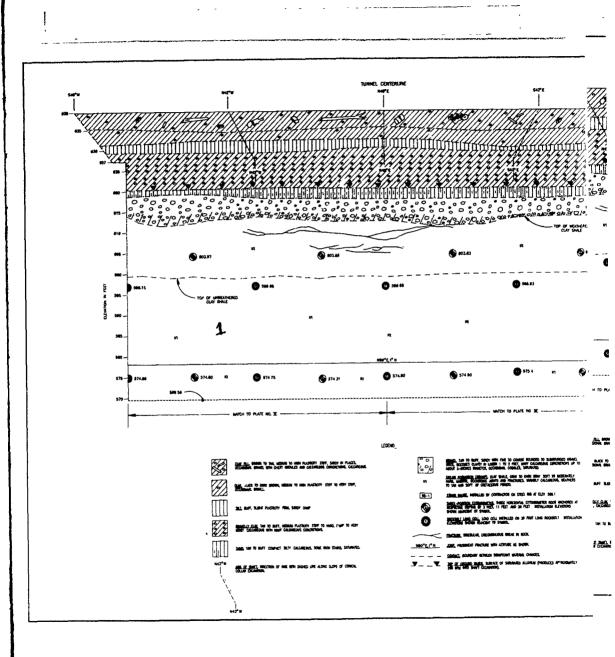
Plates (maps)

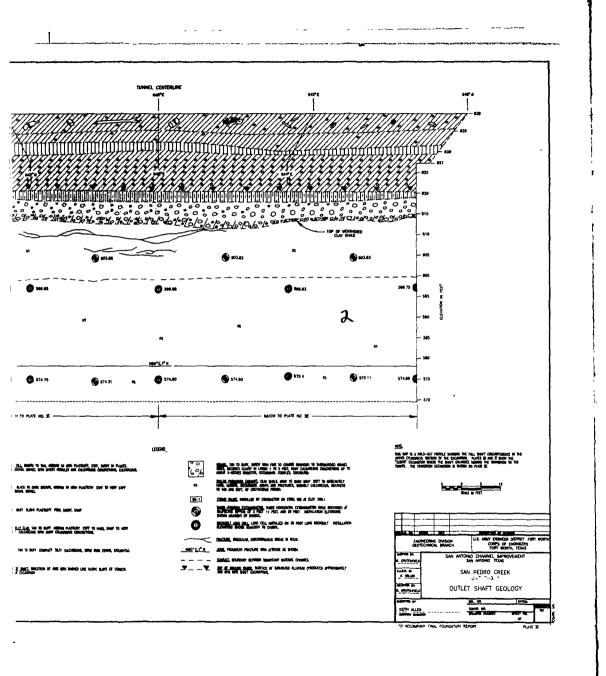


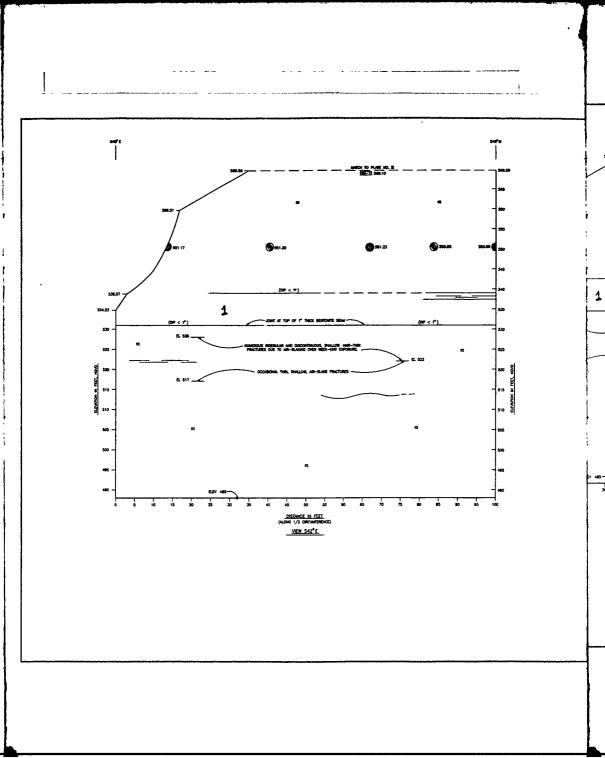


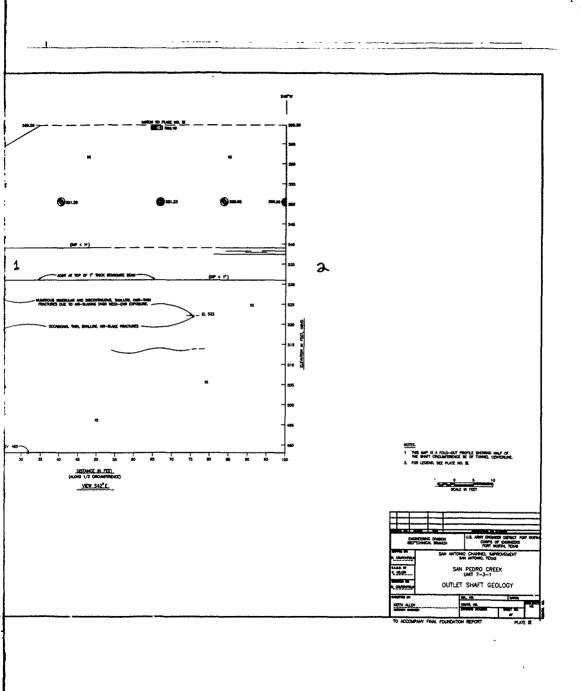


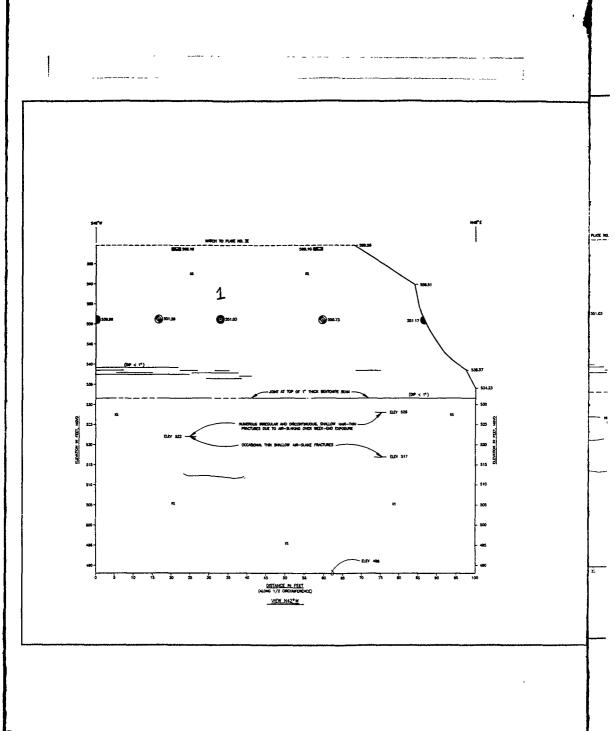
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TUNNEL ALIGNMENT

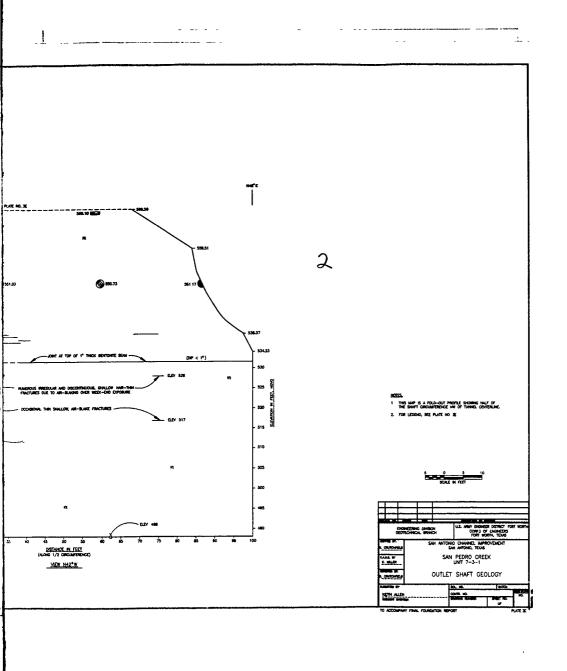


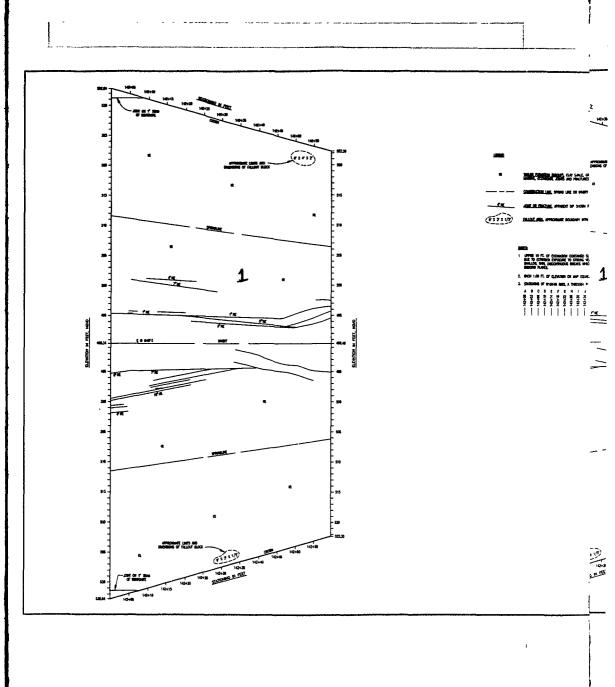


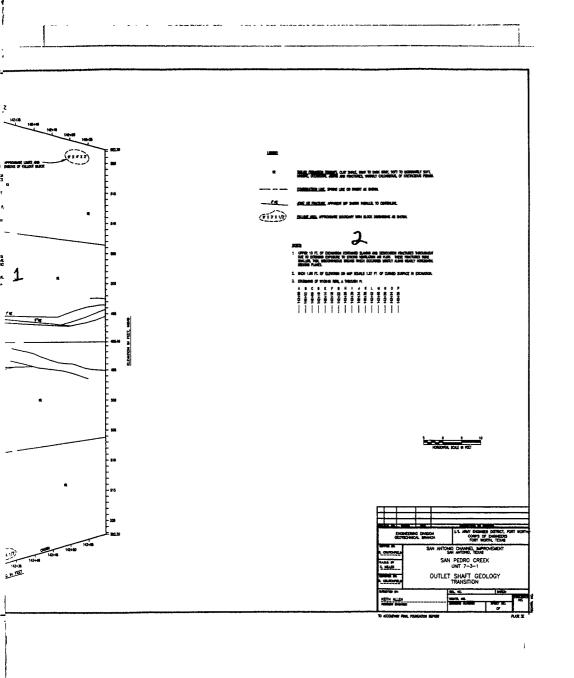


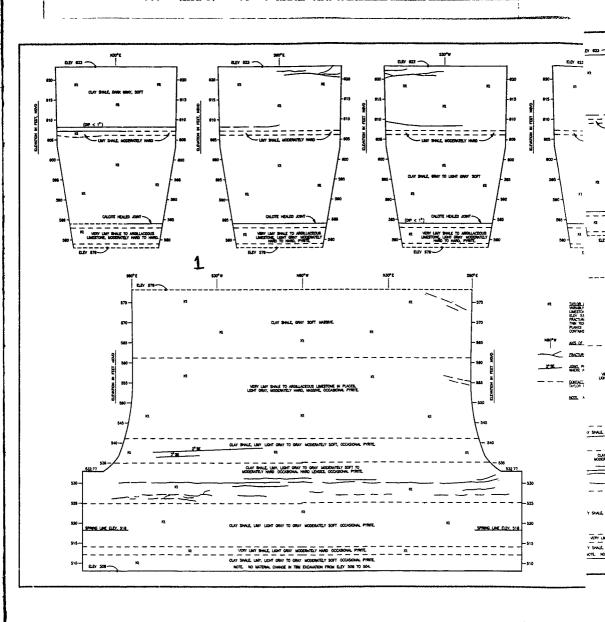












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APPENDIX F

Boring Logs (design investigations)

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	▏∃		very stift, dru to "! noist, brown % relies	frht]	.		2, 348; A. C.et in .C. L. 2.C. in 7.5; P. 3.5; in .1; B. full to 11.0; b. 11.C. in L. r. P. 16,0; in 7.1;
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			PLASTICITY, CALC,	SCAT		Ľ	J 135.1	√5 < 4 '	
	=		GRAYON WITH FI	ve to		۵	4 135:1	75'	
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	-3		GRAYEL , SEITE S	~"			200 / 100	2.120	
i	Ξ		40'-45' POCKET			4	D8 2 /2	0:135'	
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	, J		BO-100' DX BEN-GR	AY.			C3 337.3	18	
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i	∄		(BEICK & CONCRE FERSS)	7E	£ 03°	_			~>
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ı	#		MOIST, SAFF, HIGH FLASTICITY, FEW : FINE COCOS NODE	SCAT	27.0	,	20.40	SET B CASH SET B CASH DENISON NOTE DEBC FROM FLINE RUN DENISON AND RODE AND RODE AND RODE AND RODE AND RODE AND RODE	ıs
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			60C-237		-			
		Sun	73	10 ELES	AT18# 49		ten 1 des Rei	nacks
	- 67 100		040 PROUVERT	N	110LE	3	mayer 1	BMPY81
OD *****				17 ELEV	ATION TO	P OF HOL	2 40 W21	
THE				10 101	L COME N	ECOVERY	FOR DONING	1001
TOTAL D			48.0'	19 51641	A (Q)	MSPECT	Colver	
EVATION	96978	reseme	CLASSIFICATION OF MATERIA		S CONT	POT OR		nes
MATTER S	again.	CEOGNO	(Prescription)	ľ	EAV.	***	(DyBling time, and profinites, etc.	II olgalitic and
12/	ao -	-	00'401'			1		
	=		ASPHALT				1 SAMOLES	<u> </u>
	۱ -	l i	01'10/.2'				JARS	
		!	BASE COURSE/GE	arı			A 01-12 B 12-18	?
] =		THEY, DRY, MED DE	VSE,		c	0 18-18	
	=		THY, DRY, MED. DE FINE & COMMISE OF YERY SAMOY	ewi,		-	0 18-48	,
	1 =		12'40 9.0'				E @ 100	
	hadaalaa		FILL/CLAY			oxdot	G-115:15	oʻ
	=		12'-18' OF BON, DA	me.		ا د ا	H 150:-16	o .
	-		MED STIFF, MED	N FINE		200.7	1 100-11	U
	=]	Coco Novices, 3	70.	i	- 1	DENISONS	
	/o =		MED STIFF, MED THANTISTY, MED THANTISTY, MAN GEORGINOUSES, SE COMMESS CHOCKIN SOME MED GRA	VEL-		€ D# 2	28 / 80 - /	00'
] =		NEWOANT FINE	m/		-		
] =		COMESE MOD GR	1050			CARTONS	
	=		Genvez.			l . I	0/2/5.22 03 281-25	? 4'
	=	1	MOST, STIFF, MI HIGH PLASTICITY	%		4	C1 32 4 . 3	24'
	=		HIGH PLASTICITY	1.			C4 38 6-39	7.5"
	150'	=				//	C5 46 0'- 4	49'
	-		NOOVERS; SCAT IN GRAVEL, JOAT D (BRICK & GLASS FA	EBE U				
	=		(BRICK & GLASS FA	PA65)		r		
	=		90:0115			-	2 DRILLING	
	=		CLAY			L		00'-80'
	A =		MOIST MED STOMED AND PLASTICIT	FF,	i '		80'6"	ENUSON .
	7		MED PLASTICIT	, -	NO.	$\angle \Delta$	80'-115' CASING 10	
	3		MED PLASTIC! SANOY, ABUNDA COCO NODULES, SHELLS, SCAT GA NIGHLY CALC.	SCAT				
	-		SHELLS, SCAT GA	AVEL,	201	E'_	RUGER T	CLEAN OUT
	=	= 2	100'-115' BREENE Y	CLLOW.	1 07	B o⊀	190-210	O' BAILED
	-		BAN, VERY MOIS MED STIFF, HIS PLASTICITY, AGU	3	1	۱ ' ا	310'-48 BORNO	
	1 3		PLASTICITY, ABU	NOW	250	1	BOTTOM	
	<u> </u>		MED STIFF HIS PLASTICITY AREA GECG DEPOSITS, I DANT MED TO BO GRAVEL, SAFT O NOOVLES, VERY BRAVELLY 115' ALS O'	rasi Alse		<u> </u>		
] =		GRAVEZ, SCAT C	eco,	100	201	4	
	[=		BRAVELLY		Į	``ا	3 WATER A	
	=				293	2	AFTER P	ULLING AUGERING
			GRAVEL TANK W	WITE,			10150;0	WATER
	*		MED TO COMESE	NSE,	د ا	į .	LEYEZU	V15€102"
	:		PREDOMINAMIZ	. y	601		ENTERIA	O WATER
	_	\equiv	CPCOS, IN GREEN HIGH PLASTION	i • 601)	1	١	102'30	MINUTES
	:		LIMEYCLAY, VER	· ·	33 0'	1 20	LATER U	AS @98"
	۱ -		acryay	•	[Box .		
	:		150.40480		101	*	GUSHINA	IN FROM
	=		SHALE.		1	 	1 <i>CRSED</i> 7	PARTION OF
	1 -		15.0'-16 0' YELLOW .	seva cer	370'	801		
	1 3		YERY HIGHLY LUEN	rviter	1	7	WATER	EVEL WAS
	1 =		GREEN GRAY, S YERY HIGHLY WAS SCAT GROW NOOD SCAT FINE GRA SU SANOY	VEZ.	1	_ ا	@100'	LAS PULLED LEVEL WAS 14 HES BAILING
	. :		JU SMOY		601		WATER	EVEL WAS
G FOR	170		1 _		L	10000	EDEO CHEEK	100 t 100
- runt	· 1836	PREVIO	US 281710WS ARE 8860LETE		1	×44.71	the state of the s	1 60C-23

Bank :	LING LO	- 1	WALE.	340		Later	- ~-	Hole No 602 237	
PREJECT	SAVA	WYON	0 8444	WEL IMPROJEMEN	97	APP TTP		OF 2 SHEEL OF THE	
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443	LE RE				- 17 GA	स्था स	FF 661	GULTING OF BAICE	
			e-c		19 701	4.72.55	even.	FRILING 1500	
	14. mar.			600-237	— —			<u> </u>	_
NAME OF		500	~		10. ELG	PL MUMBE VATION OF	A CORE	HOTEL & MARKS	_
	-					E 1004.6	707	A129 COUPLETED	-
Ø-6-14				040 ********		VATION TO			-
PEPTH DA				15 D'	W 707	AL COME I	ecoves	7 700 000mm4 /00	,
TOTAL DE				100	- " "	Ocer	w	X Colver	
LEVATION	-	LEGRAN	۰	LAMPICATION OF MATER		S CORE	DOT OF	REMARKS (Depling time, open loss, depth of openingles, are, it significand	_
•	-					env.	7	(Ording that, open lose, depth of manifesting, arc. If significant	_
	1 7		14	0'-2/0' YELLOW-1	seve	80.		ļ	
]]			VERY NIGHLY W	cit-		Box.	N.BASE OF WEATHERING	,
1]		1	O'- 2/O' YELDOW-A GREEN GONY, SO YERY MARKED, FEW S FINE COCCE NOW SW SANNOY	UKES.	100	3	BASE OF WEATHERING	
1	E		٠,	SU STINOY			ì	@387'	
	=		4/	GEEN-GRAY.	700	45.0	<u> </u>	}	
	I _∃		ĺ	6-32 1' YELLOW C GEEN-GRAY, A SOFT, AIGHLY W	IDA-		No.	S OFFISET	
1	E		İ	POCKETS, SAND	y	101	25		
5941.	ت مع		32	1-381' DK BEAY	e	100		APPROXIMATELY	
""	E"]			SOFT, MOD W	54-			APPROXIMATELY 270' NORTH OF STA 170+30 % 50'L NEW LOCATION PLANE CODENNATE	
	[<u>, </u>		34					PLANE CODEDNATE	۶.
	* -		20	1:480° DX GEA NACO UNWEAR SLI SANOY 480'	HOZED,			¥ 2,160,376 Y 578,805	
	=			JU SAWOY	,		ŀ	arrest carrier	
i	1		222	780				WAT IN TO THE	
]							NEW COORDINATES WAS 10' TO THE NORTH & 4" TO THE WEST DUE TO	
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							Hole No	600-230	_
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San Ani	tonio	Channe?	Imprevenent	11 BAT	400 TTP	T 07 911	इन्डिक्ट राज्या है <i>स</i> रा		7
Carrier San Fed	iro Cn	rek, al	a. 179 + 00 o/s 50° H	1			धारका स्टब्स्टर		_
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Mulline emecre					E HOLE	7777	#11F	2 May Pl	٠
D.c					VATION 71			,	-
				16 701	AL CORE	ecover	V FOR BORING	Loo	•
TOTAL DE	PTH .	H9LE	51.5					MOLYTA	┙
LEVATION	9661H	.266#	CLASSIFICATION OF WATERIA (Proseption)	LS	RECOVE	SAMPLE NO	(Pelling the not	tes or less depth of it signaticand	
		-	0.0 to 0.1 - Amphalt.		<u> </u>	A	* Drilling 0.0 to 27 -		1
	in thin		0.1 to 6.7		}	<u> </u>	- pravelly -		Ī
	Ξ		GRAVEL - coarse to fi				ert 23' care clean out to	24.	ŧ
	<u>تــ</u>	[]	angular to round, dr	7.		В	24 to -	6" core.	ļ
	=		brown till 1.7', the brown with pockets o	n pale	Ĺ.	-	•••		ŀ
	أثب إ		nandy and clayey, hr	lers.	Γ'		Makine va	'er 4 15',	ļ
	1111111		6.7 to 20.3				Hole builed. 24 hr check "	128'	ŀ
				,			3 hr chesk	2134	į
	11111111		CLAY			c	Jaro		
	10.		6.7 to 13.2 - high/s	ed.			A. 0.1 to B. 1.2 to	1.7 6.7	ŀ
	=	1 1	plasticity, med sti meist, dark grayish enle, sandy & grave	brown		l	B. 1.7 to C. 6.7 to	6.7	ŀ
	-		cilc, sandy & grave Fill.	lly,		P	D. 11.7 to	13.2	I
	=					<u> </u>	E. 13 2 tr F. 15.0 to	15.0 20.1	ŀ
Î			13.2 to 20.3 - med/ plast, med stiff, m	high oist		E	G. 20.3 to	24.0	ŀ
	1		till 15.0", then we	tand		<u> </u>	-		ł
1	-		soft, sandy and gra	live.		l	denison bbl.	relly for	l
	11		rl cobbly, mostly o with some light gre yellowish brown.	y and		F	Carto	ns.	ŀ
	=							_	
	ت		20.3 to 51.5			l	1. 24.4 to 2. 31.7 to	32.7	ŧ
	10 —		AMENACIOUS SHALE				3. 37.4 to 4. 63.1 to	38.4	ł
	Ξ		20.3 to 37.0 - weathe	·r	ŀ	-	5. 40.9 to	50.0	ŧ
	1		stained yellowish hr and light grey to gr	CHIL		G			ł
	Ξ	7	noft to mod moft(rx	class.			Base of wes	therine e	ŀ
	Ξ	\neq	masive, calc, dry.		[凹		7	ŧ
	_		37.0 to 51.5 - unvert	thered	l.	Ī	Core store	d # Lack-	ł
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	4 67 HOLE				TE HOLE			11000:1-15-	
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PRIL	LING L	oc "	AMP. SHID	-BYAC	t Wort	1		07 5 SHEETS	
an Ped	ro Cre	ek. Sor	Antonio, Tx.	NO DIE AND TYPE OF BIT TO DATE AND TYPE OF BIT TO SATUR FOR TYPE OF					
LECATIO	4 /Course	ator or Re	/	7					
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HOLE HO	(An other			** 101	AL HO DE	OVER-		0	
		erd dri	6DC-279	14, 701	AL NUMBE VATION 6	n cont		7	
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		mcr.m so		HE DAT	E HOLE	72		Apr 1 84	
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TOTAL B		7	CLASSIFICATION OF WATTRI				1 12500 171		
EVATION .	DEPTH	resemo	(Bosorbelan)		MECOV.	POT OF	(Brilling time, opin	loss, dopph of f significand	
	=						* Drilling		
İ	=		10.0 to 21 8		1		n 0 to 10' - 1		
	=	1	SHALE - weathered, y	-1 tow	l		10 to 180' - f	" carbo-	
] =		brown, mansive, mos	: ly	1		old bit		
	I -		blocky structure wil	h a		!!			
	=	1	few scattered plast seams, soft to mode	ratalu	Ι.	ŀ	٠		
] _3	I	soft (rock classific		1		This hole		
	3		calcareous				started by gov	rernment	
	=		Open 45 degree joint				drill crew Se Jack Stoken fo	e Ing by	
	-		fracture(no slicks)	from			mation on top		
	1111	i	12 4 to 12 8', a fer healed(tight) fracti	,			Hole cased to	ten feet	
			healed(tight) fracti	***	L		and grouted in	by above	
	~ =		scattered throughous	٠.			CEPW		
	_ =	73			Lost	Box			
	-	\equiv	21.8 to 180.0		16		***		
	3	=			'		Hole to b		
	-		SHALE - unweathered, gray, massive, lime		t i		at a later de	ite	
	=	=/-	increases with depti	until					
	_		35', then remains co	neis-		lι	All core re	covery	
	7		tent until T D , mod	lerate	у	_	was wrapped I	n cheese-	
	=	7	soft until 35', then erstely hard(rock c		L30		cloth and sea wax before be		
	\exists	⇛	(fication), chemical		o]'is		in core hoxes		
	∃		after 65' to T D		replay.				
- 1	20:		green glauconitic as within shale matrix	from		3	Hole 1 catio	n Hole is	
ı	#	\Rightarrow	152.5 to 155.0'		30	ا د	87 5' at a her	ring of	
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-				16 TOTAL C	RE OF	ECOVER	PROPERTY OF	
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] =		BROWN HARD; E	uś.		98 1	DRILLING, FORING	
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•		٠-	4			<u> </u>	
	=		0.0 to 0.5" - uncrete.	ıls -			' <u>Drilling</u>
i	-		0.5 to 1.0 - Base Graw coarse to fine, damp,	red		В	0.0 to 9.5' - 7 7/F"
l	=		brown, very mandy.			•	rockbit, 0.5 to 2' - 11" drachit.
	1		1.0 to 4.0			-	0.5 to 2' - 11" drarbit. 2 to 11" - 10" arger.
	=					_	1" to 129" - 7 7/F"
	=	7	CLAY - high planticity noft to redise stiff,	·•		С	rockbit, 120 to 180' - 4" carbon core.
	=		meint, dark olive, re acattered within, por	146]8			cute.
Ì			an extremely weather		i		***
	ω·Ξ	1	shale.			Œ	No water level taken.
		#=					A bentonitie grout mix- ture sealed up hole aff-
- 1	Ξ		4.0 to 11.0				er drilling and E-log.
l	3		SHALE - badly weather	d,		R	
1	=		a most clay consister moist, some good shall			0	Hole recorded with resistivity, caliper,
	Ξ		structure after R', ; brown, some light gra	allow		C K	and passer lors by
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ļ	Ξ.		modules and concentra	tions		1	All 4" recovered core
- 1	-		11 to 120' - rockbit,			+	was wrapped with cheese.
	. =		unweathered dark gray	# *			cloth and scaled with heated wax.
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	3		120 to 1P03				
1	3	7	SHALE - unventhered, d	ark]	Hole locations
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	FORM 1-					┸	\bot				-
FORM 1836 PREVIOUS ENTITIONS ARE ORGANIZED PROJECT HOLE NO.	4 51 m 18 :	Já Pag			MOLETE	PRO	NECT.				

F-20

DRILLING LOC	SWD	-ATTAC			Note No. 6A4C-2P1
Phones	Sen Antonio,/Tx.	N 61/0	AND THE	th	
LOCATION (Condenses	- Bratism	-1			
SAILLING ASSUCY					CHATION OF BAILL
USCE	beeing state!	- " iii	61. PR. 85		14) "
		14 707	AL HUUDE	18 CORE	sorts 13
Reese of Hilyar	d drilling.		v4110# &		
Battater Cheer		ــــــــا ،	7 HOLE		6 April 84 May 194
THERMESS OF OVEROV		19 107	AL CORE	RECOVER	Y FOR BORIUS 100
PEPTH BRILLED INTO F		- 17 100	ATUNE O	INSPEC	1 When Merky
	1	HALS	1 CORE	POIL ON	RFWARES J
			***	100	(Pelling care opray less, depth of weathering, of it significant)
1 3	0.0 to 0.1 - Aephalt.		ļ	4	* Drilling
4	0,1 to 1,2			i	0.0 to 8' - 10' auger.
4	GRAVEL		l	C	0.0 to 8' - 10' surer, 8 to 120' - 11" drapbit, 120 to 180' - 4" carbon
1 =	0.1 to 0.7 - bane gr	1	1	1	core,
=	coarse to fine, med	lun		L	Slow drilling noted by riller after 130'.
 	dense, damp, white, and silty.	sandy		ֹם ו	
一上连	= 1		1	١	.,.
	0.7 to 1.2 - tase gr	avel - st,			No water level, Hele
	📥 dark brown, very cl			1	grouted up after F-log.
10	eandy.			Đ,	Hole recornd with gramm
1	∃1.2 to 5.4			3	remintivity, and callp-
	3		l	1	er.
- _≢=	CLAY -high plasticity moist, dark brown to		ľ	ŧ	.,,
	člive, slightly sam	y.		Ιì	All core recovery was wrapped with cheene-
				11	cloth and seale! with a warmed up wax.
	5.4 to 1.0				ermin oh mer-
	SHALE - badly weather	ed to	1		Hole locations
	a soft/medium stiff omeistency, yellow	brown,		Ш	Hole is 102.7' from
20'	manive, calcareous,	moist,			SF-700 at a bearing of
	=			 	S 38' W.
-=	8.0 to 120.0 - dragbit	, shale	ľ	11	
	established.	•			Jare
	120.0 to 180.6			11	A. 0.1 to 0.7
	SHALE - an unweather	eri			B. 0.7 to 1.2 C. 1.2 to 5.4
	dark gray to white,	very			D. 5,4 to 8.0
_=	limey, moderately he classification), mas	re(rock			
	pyrite lenses scatte	- 1			
30-	throughout, very pyr				Unweathered primary
" #	from 140 to 150's,				not established.
1 4	Chemical odor through	hout.			
	Green glauconite san	rd .			
	within from 158,6 to	160.5	٠ .		
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an fed:	ro Cre	ek, Sar	Antonio, Tx.	11 BAY	DE PORTE	CEAN	TIO	A TAUGH (TOD (TOSE)	
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and the custom on growing strict 6AUC-2BI				10 TOTAL HUMBER CORE BORES						
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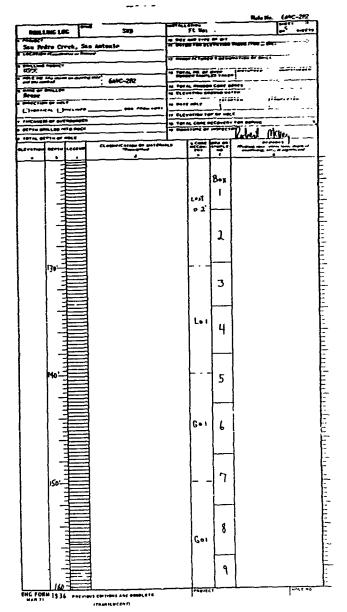
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	ro Cre	ck, Se	n Anto	nio, Tx		17 BX1	GE FOR T	EAT 419	H SHOW # 779	a 2 施订		
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HANG OF SMILLER Reese of Hillard drilling,				IS TOTAL NUMBER CORE BORES								
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13	HIGH PLASTICITY; HARD; DAMP; GI CALCAREOUS; W BRICK FRAGME	ÖLALI	: 1	ĝ	LI Sepr. 84
4	CALCAREOUS W	TH	12	<u>)</u>	1
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1-3	PLASTICITY; LIG BROWN - TAN;	HT	- 1	1627	2 Car
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1 = 1		- 1	- 1	^	4: 71.9 - 72.9
13	4.0'-10.0':MEDIUM	MARK		ł	C.E. SAMPIES; C.I.: 56.2: 57.2: 2: 41.5: -62:5 3: 47.2: -68.2: 4: 71.5: -78.5: 1: 83.1: -84.1: 1: 88.3: -89.3: 8: 94.8: -95.8: 9:100.6: -101.6:
3	BROWN; STIFE -V	ERY	- (	U	7: 88.3" -89.3"
	STIFF; SLIGHTLY AN CALCAREOUS; WITH SCATTERED BRAYE	٦		П	8: 74.8 - 95.8 9:100.6 - 101.6 10:105.7 - 104.9 11:111.6 - 112.6 12:116.8 - 117.8
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1 1 1	HIGH PLASTIFITA	344 v	- 1	11	12:116.8 - 117.8
20 -	BROWN : MEDIUM VERY MOIST : SIC CALCAREOUS	74.	. )	11	12:116.8 -117.8 13:122.6 -123.6 14:128.9 -129.9 15:134.8 -135.8
	12.5'-16.0':MEDI HIGH PLASTICITY; BROWN-TAN GR.	1UM -	- 1-1	Н	16:140.6-141.6
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	MEDIUM - STIFE; MOIST; VERY LIM WITH SCATTERE	y 7	- [ ]		18:151.9-1529 19:158 2-1592 20:162.7-1637 21:1689-1699
	GRAVEL FROM 14	0	- } ]		22:174.7 -175.7
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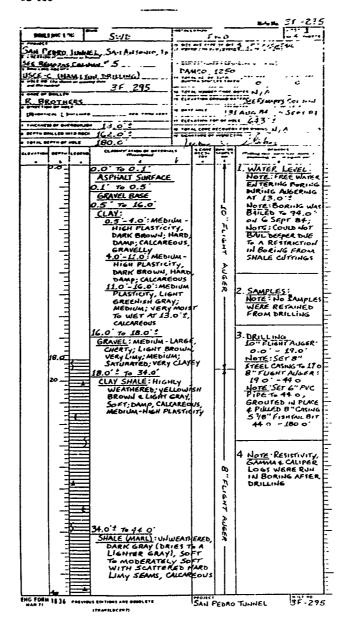
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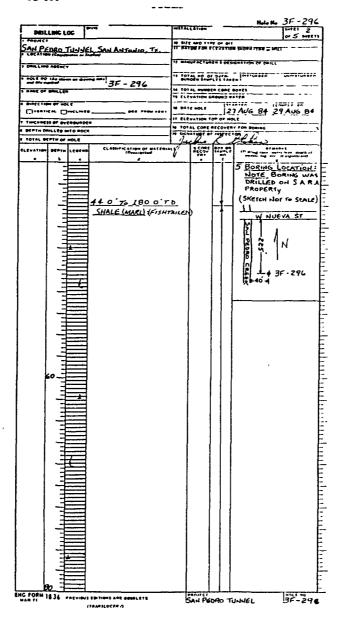
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20 -	SCATTERED GRA  16.0-19.5: MEDI PLATICITY: TAM LIGHT BROWN; WET; VERY LIM SCATTERED GRA  19.5 To 20.8: GRAVEL: FINE-MET BROWN; MEDIUM, SATURATED; CLAYE 20.8: 1 TO 33.0: CLAY SHALE: NIGH WEATHERED, YEL BROWN WITH LIG GRAY, SOFT; DAI CALCARCOUS, MEDIUM JANE GRAY, SOFT WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED, MEDIUM WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEATHERED WEAT	WELL MEDILA MEDILA MEDILA MY, WI MY MEDILA MY MEDILA MY MY MY MY MY MY MY MY MY MY MY MY MY	rii	8" Есібит Айбея	PITE TO 44 0', GROUTED IN PLACE EPULLED B"STEEL CASING 57/8" FISHTAL BIT: 44 0 - 180 0  4 NOTE: GAMMA LOG ECAL PER LOG WERE RIN IN BOR ING BI JOALIG B4 RESISTINTY PROBE DID NOT FUNCTION PROPERLY
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			WITH SCATTERE  1.4 - 4.0 : MENU  YELLOWISH BR DAMP; SILTY; VI  EOUS; WITH L	o GRA	TICIT.	(4 50)		
- 1	1	!	YELLOWISH BE	buw f	480	DB 2		
- 1	1		BAMP; SILTY; VI	gry ča	CAR-	1400)		
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į,	E_05		VERY STIFF - A SILTY; VERY CO WITH LARGE A	LIAR	COUS			
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į,	긬	- 1	VERY CALCAL	EOUS	L	[ ' ' ' ]	12.90 1 -	911.
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- 1	⋾	4	SAND: FINE GRAIN	VED;	اريدا	Α	15.1145	1155
	_7	====	LIGHT BROWN; DAMP, SILTY, WI FERRUGINOUS S	TH	31.5	<b> </b>	16 120 5	- <i>/21</i> 5
- 1	=	-4	FERRUGINOUS S	TAINS,			16 132 2	
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=	Ш	GRAMELLY CLAY) 25.0'\$ TO 29.5'\$	ı,	L:1.3	,	1.BAG SAMPLES: B-1:22.5 -25.6
-		CLAY ! RE-WOODED !	minute,	1):	2	
1 5		ME Soules - High PL	96- 1	13.5		
-	43	BROWN: VERY ST BAMP: VERY CAL GRAVELLY: (MOTE	ice:	и.	_	5. DRILLING:
3		GRAVELLY; ( NOTE	E: 04/	ه.و:)		10" FLIGHT AJGER
=		RESENTATIVE SAM	47° I	17.0		CANNE TO 4.0
3	${\mathbb H}$	DUE TO GRAVEL T			3	B" FLIGHT AUSER
3		WATER IN BORIN 29.5° = 16 39.3° =	* k	:00		(CLEAHOUT)
_=	=14	CLAY SHALF: BAD WEATHERED; ME	c	50.5		4.5 - 27.5
=		WEATHERED; ME HIGH PLASTICITY	Ebidud-			NOTE: SAMPLE WA
-=		YELLOWISH BROW	ر است	:0.1		DISTURBED FROM
=		YELLOWISH BROW WITH LIGHT GRA SOFT; DAMP; CAL	Lies	us;	4	TO GRAVEL NOTE: PULLED 4.0
13		WITH LIME POCK	ETS: 4	54.5	Ŧ	STEEL CASING, BALL
13	朌	SLIGHTLY SILTY; FERRIGINOUS SI	Aivs			BORING & REAMED WITH AUGER
13	<u> </u>	WITH LIME COK	34.3	5:0.3	-	10" FUGIT AKER:
13		& 37.6 ; HIGHLY FRACTURED WIT	وا ر	58.5		NOTE: KESET E"
1 3		WELL HEALED FI	eac-		5	30.5
₩-		TURES THROUGH		:0.0		30.5' B"FLIGHT AUGER: 30.5' - 31.5' G"CORE BARREL:
1 3	- 6					6"CORE BARREL: 31.5 - 43.5
=	Ш	SHALE: (MARL); U WEATHERED; DA GRAY (NOTE: BR TO A LIGHT BL	IES S	2.5		NOTE: PULLED B
		TO A LIGHT BLI GRAY) - VERY CA	Jish LL-			STEEL CASING BALL
3		GRAY); VERY CAREOUS; SILTY SOFT-MODERAL	. 6	· 0. Z	6	BORING, SET B"FVE CASING TO 43.5' & GROUTED IN FLACE
4		SOFT WITH OCC	AS-	4.5		AFTER ALLOWING CE
=	=7	IONAL MOBERA SOFT-MOBERAT				MENT TO HARDEN OVERNIGHT, CEMEN
1 =	$\equiv$	HARD SEAM; W	TH	5:03		PLUG WAS BRILLED
1 3		No FRACTURES O SERVED IN COR				G" CORE BARREL:
13	7	bry-damp; foss Erous; with sa	MALL	7 <i>0.</i> 5	7	43.5' - 180 0'
=	<b>■8</b>	AMOUNT OF HYS	20-			
13	===	BRILL FLUID; w	IITH P	:0.1		G. NOTE GAMMA & RESISTIVITY LOGS WERE RUN'IN BOR
=		OCCASIONAL PY		24.5		WERE RUN'IN BOR
=		37,6'-40 6 : L 44.3 : LIMY 45.3'-48.6': CRACKS SUGH	My		8	
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3		UPON EXPOSU	7E			
=	=9	48.6' - 51.6 : M	08-	785		
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		51.6-84.7:50 CRACKS SLIGH		L:0.0	9		
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- 1		FINE SAND		L:0.0			
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- 1		LIGHT GRAY E SLIGHTLY WAS 108.9'-114.0:	AND.	L:0.0	14		
Ì		MODERATELY	SOFT	110.5			
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- {		114.0'-118.4	SOF		15		
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HG FORM 1836 PREVIOUS EDITIONS ARE OBSOLETS SAN PENRO TUJUKL GASC - 30		크						<b> </b>					E_
HG FORM 1836 PREVIOUS ENTIRES ARE OBSOLETE SAN PENRO TUJUKL GASC - 30		=		l				•					E
HIG FORM 1836 PREVIOUS EDITIONS ARE OBSOLETE SAN PENDE TUNNEL GASC. 30		亅		1				1					E
HIG FORM 1836 PREVIOUS EDITIONS ARE OSCILLES SAN PEDRO TUNNEL GASC 30		, ∃		l									E
MARTI SAN PENRO TUNNEL GASC-30	NG FORM	1836	PREVIO		TIONS ARE	9996\ E78		200121	L			77.5	
	MAR 71							J PVC.	t hRo	TUNNE	L	GAGO	- 105

DL-22	25							Mala Mar (Dura and		
0001	LINE LO	_  -	,50	-17-51	Átres	-		Hule Ha. 6D4C = 306 eerth seety j	ı	
PROMET				10 MIE	400 TYP			4" shriby Tube/4" Core Bar	٠,	
E. LOČATIO		Antoni	o, Texas	11 827	PON E	LUV	TIO	AND SAME OF THE PARTY.	``	
Sni	edro C	reek 1	urrel	15 WARDY ACTURES 5 OF SIGNATION OF BRILL						
Humilto	n Oril	lin-	nd Engineer Testing	Desco 1250						
	4:5		604C - 506		AL MUMBE		_			
		Dua	ne Brothers	19 EFE	YATION 0	į	••	ATER See comment i		
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1 THERMEI					VATION T		***			
DEPTH 0					AL COME O			7 FOR BORING ( 1		
- 10141 00			31.0	Ļ-			_	hander hy kede	Ł	
ErdAque.	96614	L269=0	CLASSIFICATION OF MATERIA		e cont	145	77	(Prilling House, majors from draph of mountaining, one. If sugnificant		
	ᆣ		0.0' to 6.0'		<del>                                     </del>	7	-	· Viter Level	F	
	1 =		CL\Y:			F		Briled the hole, Tre hole gaved just below	Ε	
	-3		0.0' to 0.5's low	e liv	ŀ	<del> </del> -	_	the hottom of the car-	E	
1	3		plimiticity; gray; t dry; allty; sande;	cil.	1	2	_	ing whom the water was removed. 2" hr. check-	Ē	
	$\equiv$		careous.			1		witer at 7.7° and nave	Ε	
	]		U.j' to 2.0's med: high planticity; b	ium-	Ì	۴		below 10.1'.	E	
		K	-tiff; dry; milty;	sandy	1				Ξ	
	=	<del>/</del>	prively; raicireou			E	-	2. <u>Jar Sumplem</u> A. 0.0 - 0.5	Ē	
	Ξ	<del>-,</del>	2,0° to 3,5°s medi high pl-ticity; ye	llov	9.0	ļ.,	=	B. 0.5 - 2.0	Ë	
	10		brown; medium stiff; privally; sund/; ca	mois		П	8	C. 2.0 - 3.5 D. 3.5 - 6.0	Ē	
- 1	" =		BONE.	11-11-	444		U.	l E. 6.0 - ^.^ [	Ξ	
- 1		Ш	3.5' to 6.0's blob	l ıs-	føn L	Ш	_	F. 14.4 - 14.6 G. 10.0 - 10.2	Ξ.	
	Ξ	7=	ticitvi veilow brow gray; stifi; solst;		L-0.0		1	11. 27.6 - 25.0 1. 27.0 - 27.2	Ē	
		+	KA barran			П	′		Ė	
- 1	=	$\mathcal{I}$	6.0° to 51.0°		Kun ?	F			F	
į		7	CLAY SH. LE1		4// 5	П		5. Shelby Tube Pushes 1. 0.1 - 1.5	=	
	-	$\Rightarrow$	6.0' to 17.9'		1-0.0		В	2. refu il 3. '.0 = 4.5	Ē	
- 1	-	=	yellow brown; mot weathered; mility;	cal-	19.0	Ξ	6	4. 4.5 - 5.5	Ε	
- 1	1	1	Carenus; numerous	,	RUNY	1	X	7. 5.7 - 7.0 6. 7.1 - 11.1	Ξ	
- 1	20 -	==	alightly for allif	'roue.	1			for a least the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formation of the formati	;=- /=	
Ī	E	#	17.9° to 18.8°					materials from the	Ξ	
J	=	===	hrown- rains mod	erate	7 23.0	7		tube sumpler, with the rest of the sumple	F	
1	E		hards milt/s ca	nwi Tust-	€ W/1 K		7	discarded.	Ē.	
1	=		18.8' to 21.9'		L-0.3	١١		) l	Ξ	
	4		ten-brown (alte					4. Brilling Hethods	-	
	=	=	ing zones); sof moderntely hard	1- 1	2.70 fun 6		_	0.0 - 1.5 - /" nelby tube	Ē	
	- 크	$\equiv$	eilty; ralcareou				ķ	1.50 - 10'	=	
	=	_	20.6 - 21.8 - vertical healed	frac	1-06		,	1.0 = 1.0 = 1	Ē	
	o크		ture.		٠, , ,		3	shelby tube. Resmed the ble with	Ē	
l	=		21.9' to 31.0'	.	sue.	┌┤	-	a 10" auger to 8.0"	Ξ	
	-		gray; moderat hord: silty;	elv				and met 8° of 8" steel casin",	Ξ	
- 1	=		chlohreous. Unweithered.					Cleaned out the hote	Ē	
			ollar Tellalian*					0.45-2.	<u>-</u>	
								9.0 - 31.0 - 4"	Ξ	
- 1	===		T.D. 51.0'					Riled the hole and	Ξ	
1	1							milind all of the creings	Ē	
	=							Grouted the hole with	-	
HC BOR	تيا	L	L		P80/971	L		a bentonite/nest grow	_	
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PROJECT				to the and twee or art A" Sir-lby Tube/4" Core In							
1 165 114	- ian	<u>inteni</u>	o, Texas	11. BYANG ARE SCEARAGE SHOOK SLINE S. 1865)							
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-								M100  C+		1	
(g) -1 - 1 -	دمد (C).	mc r 44				_	117	April 86 18	Airil 95		
* ***C##**	1 07 OVE	****	6,8		VATION T			T FOR 900H40	83 .	-1	
	IILLED II	T 0 AGC	21,2	19 2001	AL COME	-	-60	190 ()	:	<u>-</u> [	
O TOTAL DE	EPTH 07	HOLE	26.0	<u> </u>					i urays	J	
CLEVATION	96514	reema	CLASSIFICATION OF MATERIA	LS	uccon-	201		(Delithing ripel, water continued, and	es total depth of	1	
		_•				L.	7		olgistic and	1.	
	-		0.0' to 6.4'			Г	1	1. Water L	vel	F	
	1 3		CF.X:		1	Ľ	<u> </u>	# Iled the	hole. 48	E	
,	-				Į.	l		hr. check	- VITOT	E	
	$\exists$		<u>U.O'</u> to <u>1.5</u> '; medi planticity; cray; c	+(ff.	r	۱,	`	at 6.8' on below 11.7	d caved	E	
ļ '	L		eli-htly moiet; sil	tvi	l	i	5	"."	-	L	
	=		sandy; oatcarenus;	tr're	l	l				F	
			of ;r vel.		ì	H	_	, <u>jazzu</u>	100	F	
	=		1.5' to 5.5'1 redi	um-lii	h	Ľ	-	A. 0.0 - P. 1.5 -		F	
	=	7	pl sticity; tur-bro	ALC:	ł	г-1	-	C. 5.5 -	6.4	F	
	4	7	stiff; damp; milty;	ETM!	ł'	l		D. 6.4 -	<b>4.</b> ₽	F-	
	∓	<b>/</b>	gravelly, very erry below 3.0' with ale	st #1	en .	l	-	E. 6.8 -		-	
	7	$\overline{}$	up to 3.5"; c-lcare	ous.	100	ı		F. 14.0 - G. 17.9 -	14.7	F	
	10-	$\Rightarrow$	5.5' to 6.4't lov	el is-	(31.1	г	A	H. 27.6 -	27.0	F	
	7		ticity: very licht	マル	L - 4.8		ľ			F	
	<b>-</b> 7		median tills moist	:	/2-e_		4			F	
]	=	7-	nbundant lime nosul pilty.	661	Env 5		X			E	
	_∃	4	-			L	^	5. Drilling	Mothoda	E	
	$\exists$	$\vdash$	6.4' to 6.8'		L-1,5	4		0.0 - 7.5	- 4" shell	Æ	
[	∃		GAMD: tang fine gra		[,,	ĺ	ш	tube.		E	
	-		medium den e; moist;	Tome	Mary 1			7.5' to 10.	n <b>- 8</b> "	Ŀ	
	<b>=</b>	$\pm$	gravel.		14113			umer. 10.0 - 16.0	~ 4° cos	.l:	
1	_=	==	6.8° to 28.0°		1.00		H	horrel.	- 11 1177	E	
- 1	#	==	CLAY SHALE:			-	B	Rearred the 1	hale to 1 .	12	
- 1	#	==	6.8' to 10.0'1 yel	lov	200		0	oliced 10'	urer and	E	
	~ <del>-</del>	=	brown-erry; soft; h	irtely	Rynu		X	c ring.	OI II" -tee!	-	
	‡		weathered; milty; c	ile ir			^	Cleaned mit		]=	
- 1	_=		enunt a few lime nowle	suter	L. O. 6		,	to 17.0" vi	to o Am	1-	
I	#		Numerous healed fra	••			FI	17 0 4 20	0 4	E	
1	7	==	10.0° to 17.8° ve		27.0		8	17.0 to 2" harrel.	." - 4" '	E.	
	7		brown; soft; we the		RIM5	l	0	Buled t c	hole and	E	
i	_ ∃		siltr; culcarenus;	c=l-			x	pulled all		Æ	
1	弖		c renum. A few colc		L 0, 2		^	ing.		1-	
	∃	=	filled fractures. F 10.0 = 10.7 = hea	led			ا را	Grouted t'e		,I=	
I	<u>-</u>	==	vertical fracture.		28.0	Æ	3	mix after t		E	
ĺ	$\exists$		Iron nodule at 16.6	۰. ا	7		1	level was "		=	
	= =	1	17.8' to 28.0'1 FT	,y,			-			1	
ì	50 <u></u>		moft-moder tely her	1:			- 1	4. Shelby T	ibe Tuebes	1	
ſ	" 🗐	- 1	unweithered (Iron 9	trinod			- 1	1. 0.0 - 1	-5	E	
i	ᅼ	- {	nt in.8'); silty; c	sic st	1		- [	2. 1.5 - 3. 3. 3.0 - 4	.5	1	
Į	= =	1	eous.	İ			- [	4. 4.5 - 6	۰.0	1=	
	⇉							5. 6.0 - 7	•5	1-	
ĺ	ㅋ	ſ	T.D. 28.01	ļ	i !		- 1	Regresentativ	ון תרפה צרן	F	
1	= =		****	- 1			- 1	of the materi	h the re-	F	
	ㅋ	1		- 1				obtained, with	. sne zem * material:	-1	
1	7	l						dincarded.		-	
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MG FORM	1111				PP01761	_		L	HOLF #0	-	
MAR 71			5 COLLIONS TUE 0000F415	,	Sin P	-dr	o (*)	reek Tunnel	40AC - 3	127	
			TIPAPIL V C EPT T)		· ·		•				

DL-227							Note No. 65	1C-30		
PRILLING	Les	-	5Wb	-	ZW.	FW				
Market				- 020	A-0 110	2 00 000	4" CALFOLOY			
			- UNIT 7-3-1	1						
A, 20	2 + OO				CO_L		SHIT OF BUILD			
<u>SCE</u>	HAM	14.70	DRILLING	10. 101		OVER-		702040		
			604C-30B		4					
BROTE					V^ T100 0		OF REMAKE	ou Mad		
			984 7894 YERT	16 947	t mout		Z APRIL BL Z3AC			
MCKMEN OF			273		VATION TO	P 07 70	k.t			
CPTH COLL			7.4 ±	to TOTAL COME RECOVERY FOR BORNING 95 1						
01ML 00PTH	-		i.o '	100		P .	tree			
VAT-000 000		<b>-</b>   '	CLASSFICATION OF MATERIA	4	S COME	250	Cheffing steel, spire bee,			
· b.c	<del>: ] - :</del>	0.0	To 1.7	<u>.                                    </u>		<u> </u>	1 . /	-		
[ ]	#	C	AY: ME DIN - HASH	PLASI	icito.	Α	NOTE: NO FRE	= -		
			DARK BROWN, HAR CALCAREOUS, WITH	D; Dey	: "	В	NOTE: NO FRE WATER IN OVE BURDEN BURING	و- ا		
	#	1	CALCAREOUS, WITH L.S GRAVEL	SCAT	FRED	<u> </u>	DRILLING; BOR	2012		
	-3	1.7	16 3.0		'		WAS BAILED TO	30.c		
	3	61	AVEL: L.S MEDI	UM-LA	262.	<i>C</i>	& LEFT OPEN	ł		
- 1	4	-	AVEL: L.S MEDI ANGULAR: MEDIUM	LIGH	r ~	<b> </b>	Z4-HR OBSERVAT	104.		
7.4	E	ئم وا	BROWN; DRY, CLAS TO 7.4	·€γ	i	D	FREE WATER	LEVEL		
" <b>"</b>	_	<del>] ?,</del>	70 7.4 AV:		8.5	É	WAS AT 230	-		
- 1	$\equiv$	∄┸	AY: 3.0'-6.1': MEN NIGH PLASTIC!	- Mui			2 JAR SAMPLES	s: <b>l</b>		
	-=	∄			١		2 JAR SAMPLE: A: 0.0 1 B: 17 - 3.	7.		
- 1	=	đ	BRY-DAMP:CALC	4REOU	L. 0.0		6:17: -3.	e l		
	_	<u> </u>	6.1 -7.7 McDu	JA 27	44.		10 61 - 19			
	===	3	TICITY: LIGHT !	S ROWN	130		6:7.4 -8:	5		
- 1 .	_	∄	CALCAREOUS	14-	1	1/2	6: 135 6: 18.5			
- 1	=	∃ <i>7.</i> ≠	* 10 21.8'			1	H' 23 5'			
- 1 .		∄ ८८	AY SHALE : BAN	Y	T:10		J. 320			
		3	WEATHERED; YEL BROWN WITH LA	6NT	<b>~</b>		<u>.</u>	1		
	==		GRAV KAKTIDA	40.	18.0		3 DRILLING:			
- 1	=	₫	HIGHLY FRACTUR WITH WELL HEA	LED	1	'G"	10" FLIGHT AU	•		
2o		9	FRACTURES SLIG	MILV	L:0 0		4" SHELBY TO 3.0 - 75	<b>*</b>		
1		#	SILTY, CLAYEY; C CAREOUS, MEDICA	AL- A-HIG	, J		Note Tubes	sene.		
ļ	⋣	∄	PLASTICITY, WITH FERRUGINOUS STA	]	22.0	2	EXTRUDED 1 .	/ne [		
	#	Ħ	PERRUGINOUS STA CRACKS UPON FYE	POSJA			NOTE : EFAME	TAKEN TUO		
		3	CRACKS UPON EXP BROKEN FROM B	7.5 -	Ĺ:0.0	"H"	WITH 10 AU	ee		
	#	3,,	10 5' 8 1 10 33.0'7		0.0		ASET 8"CASI	1676		
- 1	==	₹	IALE HMARLY IL	ا عا	26.0		B FLIGHT AU 75'-85 4" CORE BARA	GER :		
	==	3 =	(ALE :(MARL): UN WEATHERED; DAY GRAY (BRIES TO L	ec			75'-85			
	=	∄ .	GRAY (DRIES TO L BULE GRAY); SOF	NSMT T:	, ,	-,"	# CORE BARR 8.5' - 33.0	Er:		
	$\equiv$	=	DRY-DAMP, SILTY	٠ ا	L:0.0	3		ŀ		
1	#	-1 (	CALCAREOUS: WIT NO FRACTURES OF	rn I	30,0			[		
	$\equiv$	∃ '	MO FRACTURES OF SERVED IN CORE	.	-147.64			į		
1		<b></b>	CRACKS Upon E	x-	L.O.O	٠,		į		
33 (	, <b>=</b>		FosuRE . 33.0		33.0			- 1		
۲,	<b>~</b> ]	٦‴۴			44.5					
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FORM 18	6 PREV	PV1 EDIT	1070 480 90001 810		PRO10/1	<u> </u>	CREEK ES	10-30		

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LING LO	-1	SWD	79 912E	AND 110	Ful	4' CACEN	on 5 meess	
ogo	CRE	K-UNIT 7-3-1	11 18211	-	TAXABL	wheel All Sale	<del></del>	
10 +	44	<del></del>						
ISCE					<u> 45</u> 0	9		
140		454C - 314				9	<u> </u>	
-			to 107AL MANDER CORE SOLES 6					
73 • 07 HOL	•				1:2	ececanic	COLVEN	
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1 av eve	ntunce	23.0			ecoves	; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ; ;	99.	
			in sensity will de micres you					
			47		907 07	PEUM	1*3	
			V	- env			it elgisticand	
0.0		00' 10 9.2'			_	I WATER L	4.EL:	
=		CLAY FILL:	أعسن	40.	В	BAILED To	51.0	
Ξ		TICITY: BROWN;	HARE	bRy-		& LEFT OF	£/	
=		DAMP, GRAVELLY	CALC	REOU				
3		TICITY: BROWN	IARD.	AS- MAP:	<b>C</b>			
. =		VERY GRAVELLY	CALC	<b>NEOU</b>	Ļ	WAS AT 1		
Ē		2 1 - 8.5 : MEDI	WRIS	GH.				
=		PLASTICITY, DAR	K 5R	own,		Z JAR SA	VERET.	
Ε Ε	i ;	HARD, DAMP, W	TH C	96-				
=		CARRON STAIN	5 W	TH		C: 5 1 -	71,	
ㅋ		COCASIONAL SM	ALL C	gavel	6	E. 82 -	7 2 2	
=		85'-92 Lou	-me	MUM	r	£ : 8.2 -		
7		PLASTICITY; LIG	WY A	ROWN	_	H:14 0' -	19.0	
3		CALCAREOUS W	ITH 4	enses	6	I:230 -	25.0	
===		OF SANDY SILT	[				•	
3		9.2 To 19.0 1	,			1	•	
ᆿ		7.2 - 14.0 LOW	-460	:/44	и	M. 45.0		
3		PLASTICITY, LIG	NY BA	ow√-	,,	0' 54 2	•	
큭								
Ξ		FROM 12 0; DA	MP T	,	_	3 BAG SAM	CLES:	
20-		GRAVELLY TO VE	2M 3	AJELL	BAG	B-1 . 18 0.	- 230	
∄		FROM 12 0, VE	Ry C	AL-	1			
ᆿ		CAREOUS; SILTY TRACE OF FINE	KANN	~		4 CARTON	SA APLES .	
23.43		FROM 12 o'				SAMPLES	TAREN	
	_	14.0 - 19.0 = 14	ow Pl	AS.	1			
∃	Ш	MEDIUM, WET,	ERY	230		5 DRILLING	<u> </u>	
−₹	≡	GRAVELLY: VERY	SILT	REOU		B FLIGHT A	KER:	
₹	==7	1 SAND			1	4" SHELBY	TUBE	
-		19.0° To 23 0° 1	}	ام و		2.0 - 1	30	
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## APPENDIX G

Geotechnical Instrumentation Report

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### FINAL INSTRUMENTATION REPORT FOR THE SAN PEDRO CREEK TUNNEL AND OUTLET SHAFT

### SAN ANTONIO RIVER AND SAN PEDRO CREEK TUNNELS PROJECT SAN ANTONIO, TEXAS

#### 1.0 INTRODUCTION AND AUTHORIZATION

The San Antonio River and San Pedro Creek tunnels and associated shafts are currently being constructed by Ohbayashi Corporation (Ohbayashi) under the Phase II contract of the San Antonio Channel Improvements Project. Phase II design and construction contract administration are being performed by the U. S. Army Corps of Engineers (COE). Local sponsors of the project are the San Antonio River Authority (SARA) and the City of San Antonio, Texas.

The San Pedro Creek Tunnel (SPCT) is approximately 6,000 ft long. It extends from a 24-ft finished diameter inlet shaft located in the San Pedro Creek channel near Quincy Street to a 35-ft finished diameter outlet shaft located on the west bank of the San Pedro Creek channel at Guadalupe Street. The tunnel was excavated from the outlet shaft to the inlet shaft with a tunnel boring machine (TBM), and it has a "one-pass" lining of precast concrete segments. It has an approximate 27-ft excavated diameter and a 24-ft 4-inch finished diameter.

The San Antonio River Tunnel (SART) is approximately 16,000 ft long. It extends from a 24-ft finished diameter inlet shaft located near U. S. Highway 281 (McAllister Freeway) at Brackenridge Park to a 35-ft finished diameter outlet shaft located in a bend of the San Antonio River near Roosevelt Park.

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The SART is currently under construction, and it is planned to be lined with precast concrete segments. When completed, it will also have a 24-ft 4-inch finished diameter.

The SPCT and the SART were designed to bypass floodwaters beneath the City of San Antonio, Texas. Figure 1 shows the SPCT and SART plan alignments through the city, and Figure 2 shows the tunnel profiles.

Woodward-Clyde Consultants (WCC) had a subcontract with Ohbayashi to provide specified geotechnical instrumentation services during construction of the Phase II tunnels and shafts. The purpose of the instrumentation program was to obtain data that would be used by Ohbayashi and the COF for design verification and future design applications.

This report on the instrumentation installed in the SPCT and outlet shaft has been prepared in accordance with Section 2C, Paragraph 2.3(11) of the project specifications, and COE Letter CW-0896, dated 17 November 1989. It is organized into two volumes: Volume I contains the text, tables, and figures of the report, and Volume II contains the report appendices.

Following this introductory Section 1.0, Section 2.0 describes the types of instruments installed, and Section 3.0 discusses the instrumentation locations and installation procedures. Section 4.0 presents the instrumentation data and Section 5.0 provides summary interpretations of the data.

The appendices contained in Volume II are as follows:

Appendix A: Manufacturer's Brochures for Selected
Instrumentation



Appendix B: Results of Pull-Tests Performed on Instrumented Non-Structural Rock Bolts

Appendix C: Records of Data from Borescope Observations

and Measurements

Appendix D: Tabulations of Raw and Reduced Instrumentation

Data

Appendix E: Plots of Reduced Instrumentation Data

### 2.0 TYPES OF INSTRUMENTATION

Instrumentation installed for the SPCT and outlet shaft consists of multi-position borehole extensometers, rock bolt load cells, total pressure cells, reinforced concrete strain meters, convergence reference points, and displacement markers. In addition, as part of the tunnel instrumentation program, bcrescope observations were made in borings drilled for that purpose in the rock mass surrounding the SPCT excavation.

With the exception of the displacement markers and the borescope, all of the instrumentation installed for the SPCT and outlet shaft was supplied by Geokon, Inc. (Geokon), of Lebanon, New Hampshire. Copies of Geokon's brochures describing the instruments that were installed are contained in Appendix A. The displacement markers were procured from Alamo Iron Works in San Antonio, Texas, and the borescope was purchased from Hocker Inc., of Houston, Texas. Summary listings of instrument types and installation locations are provided in Tables 1 and 2. The following paragraphs briefly describe the different types of instruments.

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### 2.1 Multi-position Borehole Extensometers

Multi-position borehole extensometers are used to monitor ground deformations at different distances (positions) from an assumed fixed reference position. For the SPCT project, six-position borehole extensometers were used to monitor vertical deformations of the rock mass overlying the tunnel, and three-position borehole extensometers were used to monitor horizontal deformations of the rock mass surrounding the outlet shaft.

The borehole extensometers that were installed in the SPCT and cutlet shaft were Geokon's model A-3-1500 borehole extensometers (see Appendix A). These are tensioned rod-type extensometers with groutable anchors. The reference heads are equipped with linear potentiometers for electronic monitoring of the anchor movements. The movements were also occasionally manually monitored using a digital depth micrometer. The potentiometers had a range of 4 inches, and the electrical read-out box provided a resolution of 0.001 inch. The resolution of the digital depth micrometer was also 0.001 inches, but the readings were only accurate to within about 0.005 inches under the most favorable monitoring conditions.

### 2.2 Rock Bolt Load Cells

The load cells installed for the SPCT project were "donut"-shaped, or hollow-centered, for installation on the ends of non-structural rock bolts. The load cells utilized for the project were Geokon's model 4900-50-1.0 50-ton capacity vibrating wire load cells (see Appendix A). The sensing element of these load



cells consisted of four vibrating wire strain gages. Load development measured by the load cells was monitored electronically.

## 2.3 Total Pressure Cells

Total pressure cells were installed on the outside of selected precast concrete liner segments for monitoring the development of stresses in the segments as a result of rock mass loadings. They were also cast into the final concrete liner of the outlet shaft to monitor stresses developing in the liner. The pressure cells utilized for the SPCT project were Geokon's model 4800E total pressure cells (see Appendix A). These cells consisted of two circular stainless steel plates welded together at their periphery, and spaced apart by a narrow cavity filled with an antifreeze solution. Pressures acting on a cell forces the fluid against a diaphragm, which acts against a vibrating wire pressure transducer that converts the pressure to an electrical signal.

#### 2.4 Reinforced Concrete Strain Meters

Reinforced concrete strain meters were embedded in selected precast concrete liner segments to measure the stresses developing in the reinforcing steel of the segments. The strain meters used were Geokon's model 4911 "Sister Bars" (see Appendix A). These instruments were essentially vibrating wire strain gages fixed to short lengths of reinforcing steel. For installation, the instruments were tie-wired to the reinforcing steel "cage" of the selected liner segments.

## 2.5 Convergence Reference Points

Convergence reference points function as the end points for tape extensometer readings. The tape extensometer readings were used to monitor changes in chord lengths inside the tunnel, and hence, closure and deformation of the tunnel section. For the SPCT project, stainless steel eye-bolts attached to groutable rebar anchors were used for the reference points, and the tape extensometer was Geokon's model 1600-1 tape extensometer (see Appendix A). The tape extensometer had a resolution of 0.001 inch, and the readings were repeatable to 0.005 inch.

## 2.6 Surface Displacement Markers

Surface displacement markers were installed along the SPCT alignment to monitor the amount of surface displacement, if any, due to tunnel construction approximately 120 ft below the ground surface. The specified surface displacement markers consisted of 4-ft long lengths of No. 6 reinforcing steel driven vertically into the ground, with the top of the rod flush with the ground surface. The elevations of the tops of the rods were monitored using optical surveying techniques. Measurements were made to the nearest 0.001 ft, and the readings were considered accurate to 0.01 ft.

## 2.7 Borescope Observations

An Instrument Technology, Inc. model 122500 battery-powered extendable borescope was procured for the SPCT instrumentation program. It was fitted with a right-angle viewing head that

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provided a 2-inch relatively undistorted field of view, and an attachment was available for mounting a 35 mm camera at the viewing end. The eye-piece was inscribed with 0.025 inch graduations.

The borescope was used to observe, photograph, and measure fracture openings, including joints and bedding planes, in the rock mass surrounding the tunnel excavation. The observations and measurements were made in seven 8-ft long 3-inch diameter boreholes drilled at various angles from the inside of the TBM at approximately equal spacing, excluding the invert, around the periphery of the tunnel at COE-specified tunnel stations. Centering devices were attached to the borescope to keep it aligned in the boreholes.

# 3.0 INSTRUMENTATION LOCATIONS AND INSTALLATION PROCEDURES

Tables 1 and 2 list the instrumentation that has been installed in the SPCT and outlet shaft, including the quantities installed, the installation locations, and the dates of installation and latest readings. Table 2 also shows the status of the different instruments as of the dates of the latest readings. Figure ** shows the instrumentation locations relative to the SPCT alignment, and Figures ** through ** are schematic sketches of typical instrumentation installations.

The following paragraphs provide selected details of the instrumentation installation procedures. Except as noted, the instrumentation installations generally conformed with the project plans and specifications.

## 3.1 Outlet Shaft Instrumentation

Instrumentation installed in the SPCT outlet shaft consists of 12 three-position borehole extensometers, 12 rock bolt load cells, and six total pressure cells. Four extensometers were installed at each of three elevations, namely approximate elevations 604 ft, 575 ft, and 550 ft (Project Datum, PD), four rock bolt load cells were installed at each of approximate elevations 596 ft, 575 ft, and 550 ft PD, and the six total pressure cells were installed at approximate elevation 562 ft PD. The installation configurations of the outlet shaft instruments are shown on Figures ** and **, and the installation procedures are discussed in the following paragraphs.

3.1.1 <u>Borehole Extensometers</u>. Figure ** shows a typical borehole extensometer installation in the outlet shaft. The extensometer anchors were at depths of approximately 3 ft, 11 ft, and 26 ft from the shaft wall. The reference head was recessed 24 inches in the wall to protect it from damage.

At each of the three instrumentation elevations, the electrical read-out cables from all four extensometers were routed along the circumference of the shaft through PVC conduit to a common point, where they were spliced to a multi-pair junction cable. The junction cable was then routed vertically through PVC conduit to the ground surface, where it terminates in a lockable watertight terminal box. At the time of this writing, the location of the extensometer terminal boxes is considered temporary; however, each terminal box is labeled, and identification for future monitoring is not expected to be problematical.

Complete installation of four extensometers at one elevation was generally a two- to three-day effort, including time to drill the boreholes. After a borehole was drilled, the extensometer rods and standpipe were inserted, and a cement-based grout was pumped through the standpipe into the borehole. The grout was given 12 hours to cure, at which time the rods were tensioned and the reference head was installed. Electronic readings were typically erratic up to 24 hours after grout placement, but manual readings were taken immediately following installation.

3.1.2 Rock Bolt Load Cells. Figure ** shows a typical rock bolt load cell installation in the outlet shaft. The load cells were installed on 39-ft long No. 8 steel reinforcing bars, 19 ft of which were anchored with a 2-part fast-setting resin grout. The 20-ft long free length was double-wrapped with asphaltic tape and cased with 2-inch diameter PVC pipe to mitigate any anchoring effects that might occur on the free length as a result of closure of the borehole. The rock bolts installed for purposes of load cell instrumentation were not considered to be structural members.

Following installation of the rock bolts, and prior to placing the load cells, the bolts were pull-tested for verification of anchor reliability. The pull-tests consisted of loading and unloading the bolts in 1-ton increments (or decrements) while monitoring bolt movements. The maximum load applied during the pull-tests was 10 tons. In all except one case, bolt anchorage appeared satisfactory, with the load-deformation behavior of the bolt installations being approximately linear. The exceptional case was a bolt installed at approximate elevation 596 ft, the anchorage of which appeared to fail at an approximate load of **

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tons. Appendix B contains the results of all of the pull-tests performed for the non-structural rock bolt installations in the SPCT outlet shaft.

The load cells were mounted on the rock bolts immediately following completion of the pull-tests. Each load cell was seated on 1-inch thick steel bearing plate, and was held in place by a was ar and nut assembly. In some cases, dry-pack (hydraulic) concrete was placed beneath the bearing plate, between the plate and the underlying rock mass, to improve the contact between the bearing plate and the rock mass, and to orient the bearing plate as nearly perpendicular to the bolt as possible. The end of the rock bolt was recessed approximately 6 inches in the shaft wall to protect the load cell from damage. There were no loads applied to the load cell at the time of installation other than the "snugging" load applied by the washer and nut assembly.

At each of the three instrumentation elevations, the electrical read-out cables from all four rock bolt load cells were routed to the ground surface in the same manner as the cables for the three-position borehole extensometers.

Complete installation of four rock bolt load cells at one elevation was generally a one- to two-day effort, including time to drill the boreholes and perform the pull-tests. After a borehole was drilled, the resin grout cartridges and the bolt were inserted into the borehole, and the bolt was spun to mix the cartridge ingredients. Approximately 15 minutes later, the bolt was pull-tested. The load cell was installed after completion of the pull-test. Electronic readings were typically erratic up to about 12 hours after load cell installation.

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The installation procedure for the instrumented rock bolts described herein represents a significant deviation from the project plans and specifications. As installed, the rock bolts have approximately 19-ft long by 1-5/8-inch diameter resinencapsulated anchors instead of the specified 10-ft long by 3-inch diameter resin-encapsulated anchors. This change was necessary because the specifications were inconsistent with industry standard products and procedures.

3.1.3 <u>Total Pressure Cells</u>. Figure ** shows a typical total pressure cell installation in the outlet shaft. The pressure cells were attached with tie wires to reinforcing bars, then cast in the final concrete shaft liner.

Approximately 28 days after the pressure cells were installed, the cells were "repressurized" using a crimping tube mechanism designed for that purpose. Repressurization causes the cells to expand against the surrounding the concrete, and thereby fill any voids between the cells and the concrete resulting from contraction of the concrete as it cured.

The electrical read-out cables from all six total pressure cells were routed along the circumference of the shaft to a common point, then routed to the ground surface in a tied bundle. At the time of this writing, the cables terminated at individual spools temporarily located near the collar of the shaft. Within the shaft, the cables were cast in the final concrete shaft liner.

# 3.2 <u>Tunnel Instrumentation</u>

Instrumentation was installed at two stations of the SPCT, namely Station 143+75 and Station 158+47. The instrumentation that was installed at each station consisted of the following:

- o One six-position borehole extensometer;
- o One rock bolt load cell;
- o Three total pressure cells;
- o Three reinforced concrete strain meters; and
- o Six convergence reference points.

In addition, borescope observations, photographs, and measurements were made at both instrumentation stations, and surface displacement markers were installed in the vicinity of Station 143+75.

The six-position borehole extensometers and the displacement markers were installed at least 200 ft in advance of the TBM excavation. The total pressure cells and the reinforced concrete strain meters were installed with the precast concrete liner segments, approximately 67 ft behind the TBM cutter head, and the rock bolt load cells were installed and the borescope observations made immediately after installation of the instrumented segments. The convergence reference points were installed as soon as practicable after installation of the precast segments, but it was not possible to make tape extensometer measurements until the TBM trailing gear had passed the instrumentation station, approximately 400 ft behind the cutter head.

The installation configurations of the tunnel instruments are shown on Figures ** and **, and the installation procedures are discussed in the following paragraphs.

3.2.1 <u>Borehole Extensometers</u>. Figure ** shows a typical six-position borehole extensometer installation at a tunnel instrumentation station. The deepest extensometer anchor was located approximately 3 ft above the crown of the tunnel excavation, or approximately 115 ft below the ground surface, and the other 5 anchors were spaced between the deepest anchor and the weathered/unweathered shale interface. The reference head was located at the ground surface, but recessed approximately 3 ft so that its protective cover was installed flush with the surface.

The electrical read-out cable from the extensometer was routed through conduit to a watertight terminal box. For the extensometer at Station 143+75, the terminal box was temporarily located in a lockable utility shed placed on the project site for that purpose. For the extensometer at Station 153+47, the terminal box was placed in a recessed lockable rectangular valve box installed near the extensometer.

Complete installation of a six-position borehole extensometer was generally a four- to five-day effort. On the first day, 12-inch diameter PVC casing was drilled through the overburden soils and weathered shale at the extensometer location, and grouted in place. Approximately 12 hours later, after the grout had set, the boring was advanced through the unweathered shale to total depth. Rock cores were recovered from the boring, and logged for geologic characteristics. After the borehole was drilled, the extensometer rods and standpipe were inserted, and a cement-based grout was pumped through the standpipe into the

borehole. The grout cured at least 24 hours, at which time the rods were tensioned and the reference head was installed. Electronic readings were typically erratic up to 24 hours after grout was placed, but manual readings were taken immediately following installation.

- 3.2.2 Rock Bolt Load Cells. The rock bolt road cell installations varied between the two SPCT instrumentation stations. Therefore, the installations will be discussed separately.
- (a) Instrumentation Station 143+75. Figure ** shows the rock bolt load cell installation at Station 143+75 of the SPCT. It is similar to the load cell installations in the outlet shaft. The rock bolt was installed vertically in the roof of the tunnel from the top of the TBM main beam through a 12-inch diameter "block-out" that had been cast for that purpose in the liner segment. The bolt was installed vertically rather than radially because the configuration of the TBM machinery did not accommodate a radial drill rig set-up in the roof of the tunnel. The vertical installation took a significantly longer period of time than the horizontal installations in the outlet shaft due to difficulties experienced in placing the resin grout cartridges in the borehole. Grout dripped from the borehole after it was mixed until sufficient time had passed for it to completely set.

Following installation, and prior to mounting the load cell, the rock bolt was pull-tested in the manner described previously for the bolts installed in the outlet shaft. The maximum load applied during the pull-test was 8.5 tons. Although the bolt anchorage appeared to slip at an approximate 5-ton load, it

successfully held the maximum 8.5-ton load. Appendix B contains the results of the pull-test performed for the bolt installed at Station 143+75.

The load cell was mounted on the rock bolt immediately following completion of the pull-test in the manner described previously for the bolts installed in the outlet shaft. The load cell installation was recessed in the block-out cast in the 12-inch thick precast liner segment. The instrumented roc', bolt was stressed with an approximate 1-ton tensile load at the time of installation.

The electrical read-out cable for the load cell was spliced to a junction cable that ran along the outer surface of the liner to the hydraulic instrumentation shaft located at approximate Station 143+00. The junction cable was then routed to the ground surface through the hydraulic instrumentation shaft to a watertight terminal box temporarily located in a lockable utility shed placed near the collar of the shaft for that purpose.

(b) Instrumentation Station 158+47. Partially due to the difficulty experienced at Station 143+75 in placing the resin grout cartridges, the decision was made to change to a cement-based grout anchorage for the rock bolt installation at Station 158+47. The anchor length was increased from 20 to 25 ft, and the grout cured for 90 hours before the bolt was pull-tested. However, in all other respects, the installation was the same as the Station 143+75 installation. The bolt was successfully pull-tested to a maximum load of 8 tons, and it was stressed with an approximate 1-ton tensile load at the time of installation. Appendix B contains the results of the pull-test performed for the bolt installed at Station 158+47.

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The electrical read-out cable for the load cell was spliced to a junction cable that was routed to the ground surface through steel conduit attached to the inside wall of the ventilation shaft located at approximate Station 158+14. It was necessary to place the junction cable in a conduit to avoid damaging it when the ventilation shaft was subsequently used by Ohbayashi as a temporary pea gravel hopper. At the ground surface, the cable was routed through conduit to a terminal box located in the same recessed valve box previously described for the Station 158+47 six-position borehole extensometer.

3.2.3 <u>Total Pressure Cells</u>. Figure ** shows a typical total pressure cell installation on a precast concrete liner segment. The total pressure cells were epoxied in block-outs cast in the outer surface of the segments for that purpose.

At each instrumentation station, the electrical read-out cables from the pressure cells were routed around the outside of the segment ring to the crown of the tunnel where they were spliced to the same multi-pair junction cable as the rock bolt load cell. The terminal box for the total pressure cells installed at Station 143+00 was temporarily located in the lockable utility shed placed on the project site for that purpose, and the cerminal box for the total pressure cells installed at Station 158+47 was located in the same enclosure as the terminal box for the six-position borehole extensometer.

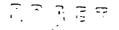
3.2.4 Reinforced Concrete Strain Meters. Figure ** shows a typical reinforced concrete strain meter installation in a precast concrete liner segment. The reinforced concrete strain meters were embedded in the precast segments when the segments were manufactured.

At each instrumentation station, the electrical read-out cables from the strain meters were routed around the outside of the segment ring in the same manner as the total pressure cells, and spliced to the same junction cable as the rock bolt load cell and the pressure cell cables. The terminal boxes for the reinforced concrete strain meters are the same as for the total pressure cells. The terminal boxes have labeled switches for the different types of instrumentation, so future monitoring is not expected to be problematical.

3.2.5 Convergence Reference Points. Figure ** shows a typical convergence reference point installation in the SPCT. The reference points were not installed in the tunnel crown or invert because the presence of the ventilation duct in the roof and the muck train tracks in the invert made it impossible to access the points for monitoring and maintenance. A 2-part resin grout was used to anchor the reference points in holes drilled in the precast concrete liner segment for that purpose.

It was not possible to make the tape extensometer measurements within about 400 ft of the TBM cutter head because the tunnel section was blocked with the TBM trailing gear and transformer jumbo. After the TBM equipment had passed the station, access to the convergence points was only available with a "man-lift", use of which blocked passage of the muck trains.

3.2.6 <u>Surface Displacement Markers</u>. Pigure ** shows the locations of the surface displacement markers installed along the SPCT alignment. The marker rods were hammered into place with a sledge. Surveys of the marker elevations were made when the markers were accessible; however, because the markers were



located in one of Ohbayashi's storage areas, they were frequently covered with stored equipment or materials, and hence, were inaccessible.

3.2.7 Borescope Observations and Measurements. Figure ** shows the locations and orientations of the boreholes drilled at COE-specified stations for the borescope observations and measurements. As indicated in the project plans and specifications, it was intended that the boreholes be drilled radially. However, the configuration of the TBM machinery did not accommodate a radial drill rig set-up in the tunnel at every boring location. The drill rig was mounted on the erector ring of the TBM.

Each boring was observed with four passes of the borescope, each pass being offset from the previous by 90 degrees. If a fracture was observed during any of the passes, its location in the boring was noted, its aperture was measured, and it was photographed. Appendix C contains the records of the borescope observations and measurements. It is noted that the borescope observations and measurements were made at only one time for each instrumentation station.

#### 4.0 INSTRUMENTATION DATA

## 4.1 Monitoring Program

During construction of the SPCT and outlet shaft, an automatic data acquisition system (ADAS) was utilized to obtain electronic instrumentation data on a daily basis. However, the data were only reported to Ohbayashi and the COE when there were significant changes, but at least once a month. The data



reports consisted of tabulations of the raw and reduced data, and plots of the reduced data versus time elapsed since instrumentation installation. In general, data reports were submitted daily for 14 to 28 days following installation, weekly for the next 28 days, and monthly thereafter through November 1989.

The instrumentation that was not monitored with the ADAS included certain three-position borehole extensometers in the outlet shaft that had become electronically non-functional (see Table 2), the convergence reference points in the tunnel, and the displacement markers. When accessible, these instruments were monitored on a daily basis for 14 to 28 days following installation, weekly for the next 28 days, and monthly thereafter through November 1989.

Appendix D contains reports of the raw and reduced data obtained from the SPCT and outlet shaft instrumentation, and Appendix E contains plots of the reduced data vs. elapsed time since instrument installation. It is noted that some of the data presented in Appendices D and E have been edited to eliminate anomalous data from the data records, and hence, the records may differ from reports that have been previously submitted.

## 4.2 Summary of Data

The following paragraphs contain brief summaries of the SPCT and outlet shaft instrumentation data. The data are summarized on the basis of shaft elevation or tunnel station. Because the different types of instruments were designed for monitoring



different ground behavior parameters, the data summaries, and subsequent data interpretations (Section 5.0) focus on the <u>trends</u> of the data rather than on the actual data values.

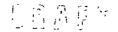
## 4.2.1 Outlet Shaft.

(a) Elevation 604. The rock mass at approximate elevation 604 at the outlet shaft site was identified by the COE as weathered Taylor Shale. It had a soft consistency, and was damp to the touch.

Data from the borehole extensometers installed at approximate elevation 604 showed the somewhat unusual trend of initially increasing in value (indicating extension of the instrument rods, or closure of the shaft section), then decreasing for a net reading on the order of +/-0.01 inch. The maximum extension measured by the elevation 604 extensometers was +/-0.05 inch.

(b) Elevation 596. The rock mass at approximate elevation 596 was identified by the COE as unweathered Taylor Shale bedrock. In the SPCT outlet shaft, it was logged as being primarily soft to moderately soft, with some limy zones.

As with the extensometers installed at approximate elevation 604, data from the rock bolt load cells installed at approximate elevation 596 showed the unusual trend of initially increasing in value (indicating tensile loading of the bolt), then decreasing. However, data from the load cells installed at positions 1, 2, and 4 indicated subsequent reloading of the bolts. At the time of the last reading, the tensile loads acting on the position 2 and 4 bolts were continuing to increase with time, although at a decreasing rate.



The maximum loads measured by the rock bolt load cells installed at approximate elevation 596 were as follows:

Position	Total Load (kips tension)	Date of Reading			
1	2.1	21 Nov. 1989			
2	4.6	24 Aug. 1990			
3	0.4	24 Aug. 1990			
4	3.8	24 Aug. 1990			

(c) Elevation 575. The majority of the displacements measured by the borehole extensometers installed at approximate elevation 575 occurred over a relatively short period of time, 20 to 30 days, followed by relatively minor to negligible additional displacements occurring over a long-term period. The maximum displacements measured by the borehole extensometers installed at approximate elevation 575 were as follows:

Position	Max. Displacement (inches extension)	Date of Reading			
A	0.086	24	Oct.	1989	
В	0.102	24	Aug.	1990	
С	0.104	24	Aug.	1990	
D	0.098	24	Aug.	1990	

The trend of the elevation 575 rock bolt load cell data was similar to the trend of the elevation 575 extensometer data, namely, the majority of the measured load development occurred over a relatively short period of time, 20 to 30 days, followed by relatively minor additional load development occurring over a long-torm period. As of the last reading of the load cell data, it appeared that the tensile loads were continuing to develop on all four bolts. The maximum measured loads were as follows:

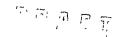
Position  1 2 3	Total Load (kips tension)	Date of Reading			
1	3.2	24 Aug. 1990			
2	3.0	24 Aug. 1990			
3	3.9	24 Aug. 1990			
4	2.7	24 Aug. 1990			

(d) Elevation 550. The trend of the data from the borehole extensometers installed at approximate elevation 550 was generally the same as the trend observed for the elevation 575 extensometer data. It is noted that the elevation 550 extensometer located at position A had been monitored manually. Therefore, there was more variation between consecutive data values than was observed for the extensometer data obtained electronically.

The maximum displacements measured by the borehole extensometers installed at approximate elevation 550 were as follows:

Position	Max. Displacement (inches extension)	Date of Reading			
A	0.092	12 Oct. 19	89		
В	0.084	21 Nov. 19	89		
С	0.063	24 Aug. 19	90		
D	0.064	24 Aug. 19	90		

The trend of the data from three of the elevation 550 rock bolt load cells was similar to the trend observed for the elevation 575 load cells, with the exception that tensile load development appears to be continuing only for the position 3 bolt. The bolt at position 1 was disturbed by construction activities, and appeared to develop a compressive loading that the load cell installation was not designed to monitor.



The maximum bolt loads measured by the elevation 550 rock bolt load cells were as follows:

Position	Total Load (kips tension)	Date of Reading			
1	Disturbed	24 Aug. 1990			
2	2.9	24 Aug. 1990			
3	4.7	24 Aug. 1990			
4	3.0	24 Aug. 1990			

## 4.1.2 Instrumentation Station 143+75.

(a) Six-Position Borehole Extensometer. Data from the sixposition borehole extensometer installed at approximate Station
143+75 indicate that significant ground movements above the
crown of the tunnel did not occur in response to the tunnel
excavation until the TBM was directly below the instrument
installation. Ground movements apparently continued to occur
for the next approximately 50 days, at which time the TBM had
advanced 443 feet beyond the instrument installation, which
approximately coincides with the time at which Ohbayashi began
placing pea gravel and grout in the annular space between the
precast concrete liner segments and the surrounding rock mass.

Data from the six-position borehole extensometer installed at approximate Station 143+75 were anomalous in that the shallower anchors 2, 3, and 4 showed greater movements than the deeper anchor 5. Furthermore, gross movement of anchor 5 did not occur until the TBM had advanced 443 feet beyond the instrument installation.

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Maximum relative movements measured by the six anchors of the borehole extensometer installed at approximate Station 143+75 were as follows:

Anchor	Max. Rel. Movement (inches extension)
1	0.0
2	0.081
3	0.105
4	0.137
5	0.029
6	0.555

- (b) Surface Displacement Markers. Survey data for the surface displacement markers installed in the vicinity of Station 143+75 indicate ground surface movements ranging from about 0.028 inch "heave" to 0.040 inch "subsidence", with no apparent consistent trend and with no apparent relationship to tunneling operations.
- (c) Rock Bolt Load Cell. Data from the rock bolt load cell installed at approximate Station 143+75 showed a reduction in load below the approximate 1-ton tensile pre-load. The most feasible explanation for these data are that the bolt anchor failed immediately upon instrumentation installation (if not sooner).
- (d) Total Pressure Cells and Reinforced Concrete Strain Meters. The total pressure cells and reinforced concrete strain meters installed at approximate Station 143+75 had erratic readings until the annular space between the precast concrete liner segments and the surrounding rock mass was filled with pea gravel and the invert was grouted. After that time, the data indicate nearly symmetrical stress development in the arch segments of the tunnel lining. Furthermore, the trends of stress development appear nearly identical as measured by the

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total pressure cells and the reinforced concrete strain meters, namely initially increasing to a maximum value, then decreasing. The maximum total pressure cell measurements were on the order of 40 to 60 psi, and the maximum reinforced concrete strain meter measurements were on the order of 3200 psi.

- (e) Convergence Reference Points. The convergence reference points installed at approximate Station 143+75 were not monitored until the TBM cutter head had advanced approximately 1000 feet (in 76 days) beyond the instrumentation station. Nonetheless, the convergence measurements indicate that the precast concrete segmental tunnel lining deformed from circular, with the diameter at the springline increasing in length, and the diameters at the quarter-points decreasing in length. The maximum measured increase of the springline diameter was 0.21 inch, and the maximum measured decrease of the quarter-point diameter was 0.07 inch.
- (f) Borescope Observations and Measurements. Very few fractures were observed with the borescope at Station 143+75, and all of the fracture observations were in borings located above the springline of the tunnel. The general orientation of the fractures appeared to be horizontal, and along bedding planes of the Taylor Shale. The observed apertures ranged from 0.02 inch to 1.6 inches wide (see Appendix C).

## 4.1.3 Instrumentation Station 158+47.

(a) Six-Position Borehole Extensometer. As at Station 143+75, data from the six-position borehole extensometer installed at approximate Station 158+47 indicate that significant ground movements above the crown of the tunnel did not occur in

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response to the tunnel excavation until the TBM was directly below the instrument installation. Ground movements apparently continued to occur for the next approximately 14 days, at which time the TBM had advanced 43 feet beyond the instrument installation, which approximately coincides with the time at which Ohbayashi began placing pea gravel in the annular space between the precast concrete liner segments and the surrounding rock mass.

Data from the six-position borehole extensometer installed at approximate Station 158+47 were anomalous in that the shallower anchors 3, 4 and 5 showed greater movements than the deeper anchor 6.

Maximum relative movements measured by the six anchors of the borehole extensometer installed at approximate Station 158+47 were as follows:

	Max. Rel. Movement
Anchor	(inches extension)
1	0.0
2	0.054
3	0.098
4	0.089
5	0.153
6	0.086

(b) Rock Bolt Load Cell. As with the rock bolt load cell installed at approximate Station 143+75, data from the rock bolt load cell installed at approximate Station 158+47 showed a reduction in load below the approximate 1-ton tensile pre-load. The most feasible explanation for these data are that the bolt anchor failed immediately upon instrumentation installation (if not sooner).

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(c) Total Pressure Cells and Reinforced Concrete Strain Meters. The total pressure cells and reinforced concrete strain meters installed at approximate Station 158+47 had erratic readings until the annular space between the precast concrete liner segments and the surrounding rock mass was filled with pea gravel and the invert was grouted. After that time, the reinforced concrete strain meter data indicate nearly symmetrical stress development in the arch segments of the tunnel lining. Furthermore, the trends of stress development appear nearly identical as measured by the total pressure cells and the reinforced concrete strain meters.

The maximum total pressure cell measurement was 113 psi, and the maximum reinforced concrete strain meter measurements were on the order of 4500 psi.

- (d) Convergence Reference Points. The convergence reference points installed at approximate Station 158+47 were not monitored until the TBM cutter head had advanced approximately 400 feet (in 20 days) beyond the instrumentation station. The convergence measurements indicate that the diameter of the precast concrete segmental tunnel lining increased at the springline and at the quarter-points. The maximum measured increase of the springline diameter was 0.13 inch, and the maximum measured increases of the quarter-point diameters were 0.04 inch and 0.12 inch.
- (e) Borescope Observations and Measurements. Very few fractures were observed with the borescope at approximate Station 158+47, and all of the fracture observations except two were in borings located above the springline of the tunnel. The general orientation of the fractures appeared to be horizontal,

and along the bedding planes of the Taylor Shale. The observed apertures ranged from 0.025 inch to 1.5 inches wide (see Appendix C).

## 5.0 INTERPRETIVE GROUND PERFORMANCE

In general, as of the dates of the latest instrumentation readings, the instrumentation data have not indicated the development of any alarming trends in rock mass behavior that would threaten the integrity of the constructed San Pedro Creek Tunnel and outlet shaft. However, it is noted that some of the instrumentation data continue to show increasin deformations, loads, and stresses. It is anticipated that the rock mass surrounding the tunnel and shaft may undergo swelling deformation when exposed to water seeping through the lining during system operation, and thereby causee additional deformation or stress development in the tunnel and shaft linings.

The following specific conclusions are made relative to the behavior of the ground during excavation of the SPCT and outlet shaft, as indicated by the instrumentation data:

o The initial peaking and subsequent decrease of deformations and load development measured by the extensometers and rock bolt load cells installed at approximate elevations 604 and 596, respectively, in the outlet shaft may be accounted for by one, or a combination, of the following scenarios: 1) desiccation and shrinkage of the rock mass on which the .tensometer heads and the load cell bearing plates are

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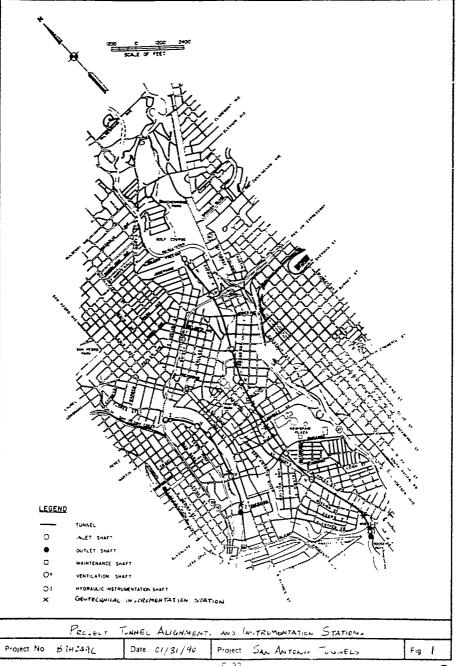
bearing; 2) the rock mass surrounding the grouted anchors of these instruments has undergone creep at the depth of the instrumentation anchors, or the anchors have failed; and/or 3) a transfer of rock loads to the shotcrete lining of the shaft. In the case of potential scenarios (1) and (2), the distance between the anchors and the extensometer head or the load cell would decrease, and the instrumentation measurements would decrease. In the case of potential scenario (3), the later increase in loads developing in the rock bolts may then be indicating that the shotcrete lining became loaded and began acting as a compression ring. Data from strain gages installed per Ohbayashi on one ring beam in the shaft excavation indicate that most stress development in the beam occurred within 120 days of placement of the beam.

- o The instrumented rock bolt installations are not compatible with either the shaft construction support system or the tunnel lining system. Therefore, the stresses calculated as being either bolt stresses or rock mass stresses are not considered to be representative of the stresses developing in the shaft and tunnel linings. Mowever, the trend of the stress development in the bolts may be indicative of the development of stresses in the linings; that is, the ongoing bolt stress development being measured by 10 of the 12 rock bolt load cell installations in the shaft may indicate continuing stress development in the shaft lining.
- o The three-position borehole extensometer measurements in the shaft appear to indicate that rock mass deformations are relatively uniform in a two-dimensional horizontal plane at a give elevation. At the time of this writing, the maximum decrease in shaft diameter indicated by the

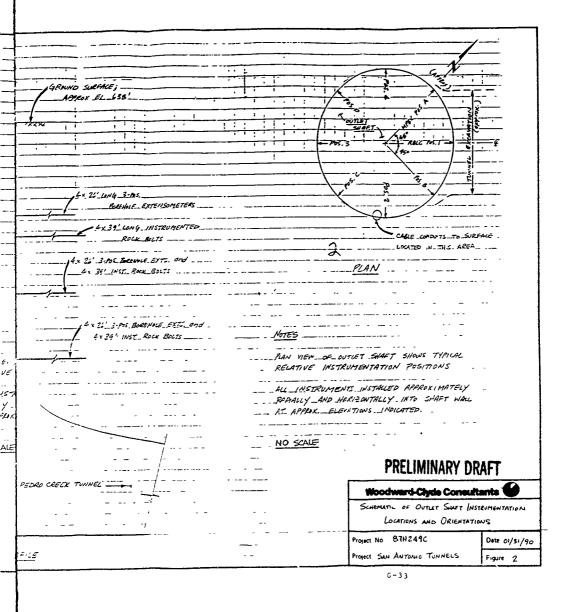
extensometer measurements is on the order of 0.15 to 0.2 inch. However, six of the 12 extensometer installations indicate that shaft rock wall movements are continuing to occur, which trend is consistent with the trend of the data from some of the rock bolt load cell installations in the shaft.

- o Data from the ref forced concrete strain meters installed at approximate Stations 143+75 and 158+47 in the tunnel indicate a relatively uniform pressure developing in the arch segments of the tunnel lining at time stations on the order of 25 psi. Data from the total pressure cells installed at the same stations indicate relatively uniform pressures developing in the arch segments ofon the tunnel lining on the order of 15 psi and 40 psi, respectively.
- o There appears to be a good correlation between the trends of lining stress development as indicated by the total pressure cell and the reinforced concrete strain meter measurements at approximate Station 143+75 and Station 158+47. However, during the course of construction, the magnitudes of the stresses indicated by the total pressure cell measurements indicated significantly greater radial pressure on the tunnel lining than did the reinforced concrete strain meter measurements. This difference may reflect the greater sensitivity of the total pressure cell measurements to relatively localized tunnel lining deformations than the reinforced concrete strain gage measurements. Such localized tunnel lining deformations could have been caused by the loads of the mining equipment operating inside the tunnel and/or differential confinement of the liner segments by the in situ rock and/or the pea gravel and grout backfill.

- o A comparison of data from the six-position borehole extensometers installed at approximate Stations 143+75 and 158+47 along the tunnel alignment with survey data from the surface displacement markers installed between Stations 143+00 and 145+00 indicates that rock mass settlements above the tunnel excavation attenuate with distance above the excavation to become practically negligible at the ground surface. It is considered that the survey data from the surface displacement markers are related to surface activities in Ohbayashi's storage area, for which there are no accurate records, and/or apparent shrink and swell activity of the overburden soils in the vicinity of Station 143+75. These effects appear to be sufficient to mask excavation-induced ground surface settlements.
- o There does not appear to be a correlation between the borescope observations of fracture frequency and orientation and other instrumentation data or observed rock mass behavior. The relatively few fractures observed with the borescope in the vicinity of Stations 143+75 and 158+47 of the tunnel would indicate that less rock mass movement should be anticipated above the tunnel excavation than was actually measured by the six-position borehole extensometers installed to within 3 ft of the tunnel crown. This finding may be due to the relatively short period of time from tunnel advance to borescope observations compared to the months-long period of time over which the extensometer data were obtained. Furthermore, the borescope observations were made through the shield of the TBM whereas most of the ground movements measured by the extensometers occurred after the shield had been advanced beyond the instrumentation station.



GRAND SURFACE; ... APPROX EL. 458'_ 1 1 7 4 × 26' LONG 3-105. ____BOREHOLE_EXTENSOMETERS_ -4x39'LENG. MISTRUMENTED___ 14.1 2 " 3-DS SCENNIE. EXTS. ON Gx 39' INST. BOCK BOLTS . EL 5751. -1 4 x 26 3- Pas BOREHOLE EXTSLAND . 4 x 39' INST ROCK BOLTS _____ --- ---· APPROX · · · · ---- PLAN VIE - RELATIVE - . FU INST - RAMALLY. - AT PPPAX NO SCALE SAN PEDRO CHEEK TUINEL SHAFT PROFILE



SMAET ROCK.	T. T. T. T. T. T. T. T. T. T. T. T. T. T	ENREMORE - (0.17.0 x 1.7. T - 2.4 x 18. town 970 SLEEVE	KOCK BOLT	12°0×12" 3°0×20° LONG (NOMINAL) . 198" 0×19° LONG (NOMINAL)	CK SOLT WAS COUPLED	NO SCALE	SCHEMATIC OF ROLE BUT LOAD CFLL (NSTALLATIONS	No B7H249C Date CX/31/90 Project SAN ANTONIO TUNNECS
		,   1 . ,   1 .		ı		'	1	Project No

Shrefere	
PRELIMINARY DRAFT	
REFERENCE HEAD GROUTABLE ANCHORE (TYP. AT 3 LOCATIONS)	
S.S. S. S. S. S. S. S. S. S. S. S. S. S.	i ! !
3	· -
Sole that t	
RESOLUTES 6. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1.	
Water	
1001G	<u> </u>
WOHINAL) BACKFILLBO - M. CEMENT GROUT	<b>.</b>
	,
NOTE SIX. POSITION BOREHOLE EXTENSOMETER INSTRUMENTION IS SIMILAR,	
3	
• INSTALLATION IS YEKTICAL, WITH KEY HEAD AT GROUND SUKFACE. • THERE ARE SIX ANCHORS PER EXTENSOMETER, INSTALLED AT	
DEPTHS INDICATED ON PROJECT PLANS.	
NO SCALE CASED WITH STEEL.	
SCHEMATIC OF BOREHOLE EXTENSOMETER IN STALLATIONS	
HECT SAN ANTONIO TOWNELS	F19. 4
	7

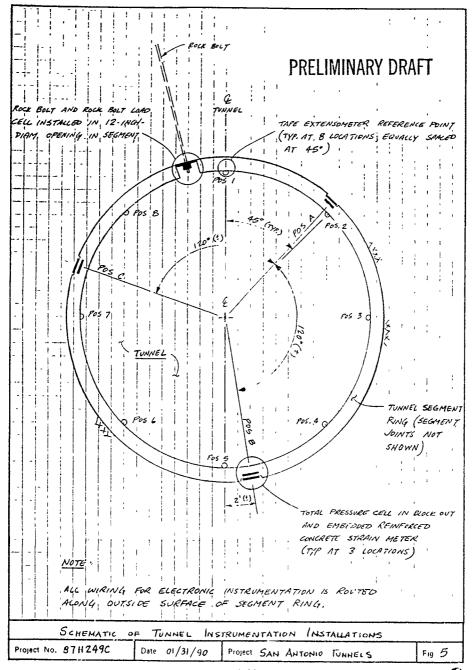


TABLE 1

San Antonio River and San Pedro Creek Tunnels Project

INSTRUMENTATION : ELEVATION (approx.)	INSTRUMENT QUANTITIES & TYPES	PRE-INSTALLATION SUBMITTAL(S)	: INSTALLATION : DATE :	INSTALLATION REPORT(S)	: RELEVANT : DATA REPORTS	SENT-FINAL READING (see note 1)	: INSTRUMENT : STATUS : (see note 2)
Elev. 604	4 - 3 position borehole extensioneters	22 Dec. '87 28 Jan. '88	3 Mar. 188	23 Mar. '88 24 Mar. '88 24 Mar. '98 28 Mar. '88	28 Har. '88 26 May '88 27 June '98 16 Aug. '98 20 Sep. '98 23 Sep. '98 21 Oct. '98 7 Feb. '89 8 June '89 25 Aug. '89	21 and 30 Nov. '89	Pos. A must be read manually
Elev. 5%	4 - rock bolt load cells	22 Dec. '87 28 Jan. '88 1 Feb. '88 19 Feb. '98 25 Feb. '98 29 Feb. '88	18-22 Mar. 198	23 Mar. '88 24 Mar. '89 24 Mar. '88 28 Mar. '88 8 June '88	28 Mar. '98 26 Hay '98 27 June '98 16 Aug. '98 20 Sep. '98 21 Oct. '98 7 Feb. '89 8 June '89 25 Aug. '89	21 Nov. 189	RELIMINARY T
Elev. 575	4 ea 3 pos. borehole extensometers, rock bolt load cells	22 Dec. '87 28 Jan. '88 1 Feb. '88 19 Feb. '88 25 Feb. '88 29 Feb. '88	8-11 Apr. '98	14 Apr. '98 20 Apr. '98 8 June '88	26 May '88 27 June '68 16 Aug. '58 20 Sep. '68 21 Oct. '68 7 Feb. '89 8 June '89 25 Aug. '89	21 and 30 Mov. '89	Pos. A exten- someter must be read manually
Elev. 550	1 ea 3 pos. borehole extensameters, rock bolt load cells	22 Dec. '87 28 Jan. '88 1 Feb. '88 19 Feb. '88 25 Feb. '88 29 Feb. '88 20 Apr. '98	6 May '88	23 May '88 26 May '88 8 June '88	26 May '88 16 June '88 27 June '88 16 Aug. '88 20 Sep. '88 21 Oct. '98 7 Feb. '89 25 Aug. '89	24 Oct. '89 and 21 Nov. '89	Pos. A exten- someter must be read manually

#### TABLE 1

San Antonio River and San Pedro Creek Tunnels Project SHA DIECHNICAL INSTRUMENTATION INSTALLED IN SAN PEDRO CREEK OUTLET SHAFT

INSTALLATION DATE	INSTALLATION REPORT (S)	; relevant ; data reports ;	SENI-FINAL READING (see note 1)	: INSTRUMENT : STATUS : (see note 2)
3 Mar. '88	23 Har. 198 24 Har. 198	28 Mar • 198 26 May 188	21 and 30 Nov. '89	Pos. A must be read manually
	24 Mar. 198 28 Mar. 198	27 June 188 16 Aug. 198 20 Sep. 188		

28 Sep. '88 21 Oct. '98

7 Feb. '89

8 June '89 25 Aug. '89

28 Mar. '98

26 May '88

27 June '98

16 Aug. '88

20 Sep. '98 21 Oct. '88 7 Feb. '89 8 June '89

SAN PEDRO CREEK TUNNEL AND OUTLET SHAFT INSTRUMENTATION REPORT. PERDING AVAILABLE ACCESS. 2. "INSTRUMENT STATUS" IS APPLICABLE

READINES.

I. "FINAL" READINGS WILL BE TAKEN PRIOR TO PREPARATION OF FINAL

AS OF THE DATE OF THE SENI-FINAL

NOTES:

21 Nov. '89

PRELIMINARY DRAFT

21 and 30 Nov. '89 Pos. A exten-

24 Oct. '89 and

21 Nov. '89

25 Aug. '89 26 May '98 14 Apr. '98 27 June '88 20 Apr. '88 8 June '98

23 Nar. '98

24 Nar. 188

24 Nar. 198

28 Mar. '88 8 June '88

23 Nay '88

26 May '88

8 June '98

19-22 Mar. '88

8-11 Apr. '88

6 May '88

16 Aug. '83 20 Sep. '98 21 Oct. '88 7 Feb. '89

8 June '89 25 Aug. '89 26 Kay '88

16 Juno '88

27 June '88 16 Aug. '88 20 Sep. '88

21 Oct. '88 7 Feb. '89 8 June '89 25 Aug. '89

Pos. A extensometer aust be

someter aust be

read manually

read manually

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Woodward-Clyde Consultants

TABLE 2 San Antonio River and San Pedro Creek Tunnels Project GEOTECHNICAL INSTRUMENTATION INSTALLED IN SAN PEDRO CREEK TUNNEL

INSTRUMENTATION STATION (approx.)	INSTRUMENT : GUNNTITIES & TYPES :	PRE-INSTALLATION SUBMITTAL(S)	INSTALLATION DATE	INSTALLATION REPORT (S)	RELEVANT DATA REPORTS	SENI-FINAL READING (see note 2)	INSTRUMENT STATUS (see note 3)	
Sta. 143+75	# 1 - 6 pos. borahole extensometer	22 Dec. '87 5 Feb. '88 6 June '88 5 July '88	13 July '88	4 Aug. 98	21 Dec. '88 21 Feb. '89 8 June '89 25 Aug. '89	21 Nov. '89		1. 4
	* 18 displacement 1	. 19 Hay '98	9 Aug. '88(?)	11 Aug. '86	21 Dec. '88 21 Feb. '89 2 Aug. '89	10 Jan. '90	8 markers have been disturbed or destroyed	2.
	* 1 - rock bolt load cell; 3 - total press- ure cells; 3 - reinf. concrete strain me- ters; 8 convergence points	13 May '88 8 June '88 6 July '88 9 Aug. '88	30 Dec. '38	7 Feb. '65	21 Feb. '89 15 Mar. '89 12 Apr. '89 8 June '89 25 Aug. '89	21 Mov. '89 and 9 Jan. '90	Pos. B strain meter cannot be read	
	* Borescope obser- vations	2 Aug. '98 12 Oct. '98	JanFeb. '89	9 Feb. '89 and 25 Har. '89	Not applicable	Not applicable	Not applicable	
Sta. 158+47	± 1 = 6 pos. borehole extensometer	5 Feb. '88 6 June '88 5 July '88	18 July '88	4 Aug. '98	26 Apr. '89 8 June '89 25 Aug. '89	21 Nov. '89		PREL'
	# 1 - rock bolt load celt; 3 - total press- ure cells; 3 - reinf. concrete strain me- ters; 6 convergence points	13 May '88 2 June '88 8 June '88 6 July '98 12 July '88 9 Aug. '88	21-67 Mar. '89	14 Apr. '89	26 Apr. '89 8 June '89 25 Aug. '89	21 Nov. '89 and 9 Jan. '90	Pos. A press. cell cannot be read	
	# Borescope obser- vations	2 Aug. '88 12 Oct. '88	Mar. '89	14 Apr. 189	Not applicable	Not applicable	Not applicable	

TI S)

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TABLE 2

San Antonio River and San Pedro Creek Tunnels Project

GEOTECHNICAL INSTRUMENTATION INSTALLED IN SAN PEDRO CREEK TUNNEL

TION S)	INSTALLATION DATE	INSTALLATION REPORT (S)	: RELEVANT : DATA REPORTS	SEMI-FINAL READING (see note 2)	INSTRUMENT STATUS (see note 3)	
0765 1. 58 P.8 S.8 S.8	13 July '88	4 Aug. 88	21 Dec. '88 21 Feb. '89 8 June '89 25 Aug. '89	21 Nov. '89		NOTES:  1. "FINAL" READINGS WILL BE TAKEN PRIOR TO PREPARATION OF FINAL SAW PERMO CHEEK TUNNEL, AND CUTLET SAWT DISTRIBUTION REPORT.
P. 8	9 Aug. '98('?)	11 Aug. 188	21 Dec. '98 21 Feb. '89 2 Aug. '89	10 Jan. '90	8 markers have been disturbed or destroyed	PERDING AMAILABLE ACCESS.  2. "INSTRUMENT STATUS" IS APPLICABLE AS OF THE DATE OF THE SEMI-FINAL
	30 Dec. '98	7 Feb. '89	21 Feb. '89 15 Mar. '89 12 Apr. '89 8 June '89 25 Aug. '89	21 Mov. '89 and 9 Jan. '90	Pos. B strain meter cannot be read	READINGS.
8 38	JanFeb. '89	9 Feb. '89 and 23 Mar. '89	Not applicable	Not applicable	Not applicable	DRAFT
EL'	18 July '98	4 Aug. '98	26 Apr. '89 8 June '89 25 Aug. '89	21 Nov. '89		PRELIMINARY DRAFT
8 8 8 58	21 <b>-2</b> 7 Mar. '89	14 Apr. '89	26 Apr. '89 8 June '89 25 Aug. '89	21 Nov. '89 and 9 Jan. '90	Pos. A press, cell cannot be read	
8 88	Mar. '89	14 Apr. '89	Not applicable	Not applicable	Not applicable	

### San Antonio River and San Pedro Creek Tunnels Project

## GEOTECHNICAL INSTRUMENTATION INSTALLED IN SAN PEDRO CREEK TUNNEL AND OUTLET SHAFT

INSTRUMENTATION Type	INSTRUMENT HODEL (see note 1)	INSTRUMENT QUANTS. & LOTATIONS (see note 2)	MOTES
Three-position Borehole Extensometer	A-3-1500	Outlet Shaft: £ 4 at Elev. 604 £ 4 at Elev. 575 £ 4 at Elev. 550	May be read electronically or eanually
Rock Bolt Load Cell	4900-50-1.0	Outlet Shaft: # 4 at Elev. 596 # 4 at Elev. 575 # 4 at Elev. 556	Load cell has 50-ton capacity
		Tunnel: # 1 at Sta. 143+75 # 1 at Sta. 158+47	
Six-Position Borehole Extensometer	A-3-1500	Tunnet: £ 1 at Sta. 143+75 £ 1 at Sta. 158+47	May be read electronically or manually
Total Pressure Cell	4800E	Tunnel:  # 3 at Sta. 143+75  # 3 at Sta. 158+47	Pos. A cell cannot be read
Reinforced Concrete Strain Meter	4911 Sister Bar	Tunnel: * 3 at Sta. 143+75 * 3 at Sta. 158+47	Pos. B meter cannot be read
Convergence Reference Points	Points: N/A Tape Extensometer: 1600-1	Tunnel: # 8 at Sta. 143+75	Invert point has been destroyed
		# 6 at Sta. 158+47	Crown and invert points were not installed
Displacement Markers	N/A	18 between Tunnel Stations 143+00 and 145+00	8 markers have been disturbed or destroyed
Borescope	Instrument Tech- nology, Inc. model 122500 (extend- able)	7 boreholes x 8 ft. long at Tunnel Stations 143+63, 143+71, 143+79, 143+87, 143+95, 158+39, 158+47, and 158+55	PRELIMINARY DRAFT

### NOTES:

^{1.} ALL INSTRUMENTS WERE MANUFACTURED BY, AND PURCHASED FROM, GEOKON, INC., EXCEPT THE BURESCOPE.  $$\rm G\!-\!39$ 

^{2.} LISTED ELEVATIONS AND STATIONS ARE APPROXIMATE.

### APPENDIX A

### REPORTS OF RAW AND REDUCED INSTRUMENTATION DATA

(NOT INCLUDED)

DRAFT FINAL INSTRUMENTATION REPORT FOR THE SAN PEDRO CREEK TUNNEL AND OUTLET SHAFT

SAN ANTONIO RIVER AND SAN PEDRO CREEK TUNNELS PROJECT WCC PROJECT NO. 87H249C

### APPENDIX B

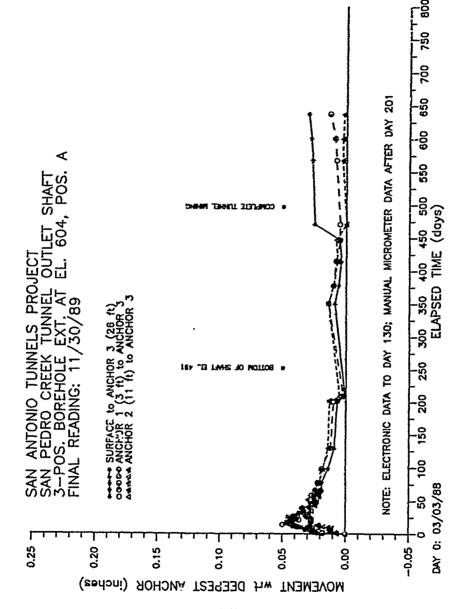
PLOTS OF REDUCED INSTRUMENTATION DATA

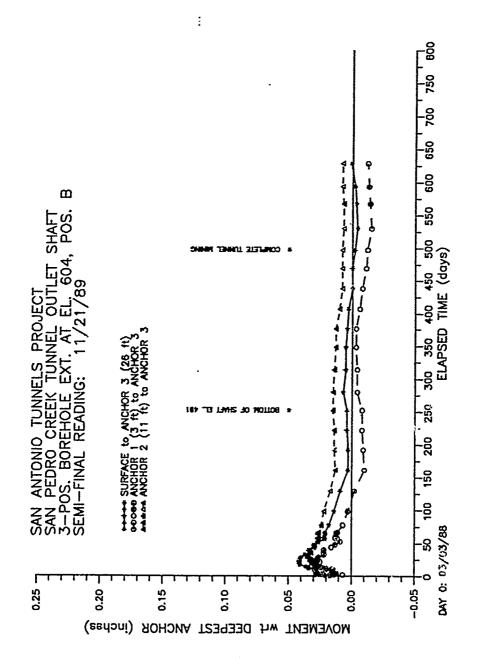
DRAFT FINAL INSTRUMENTATION REPORT FOR THE SAN PEDRO CREEK TUNNEL AND OUTLET SHAFT

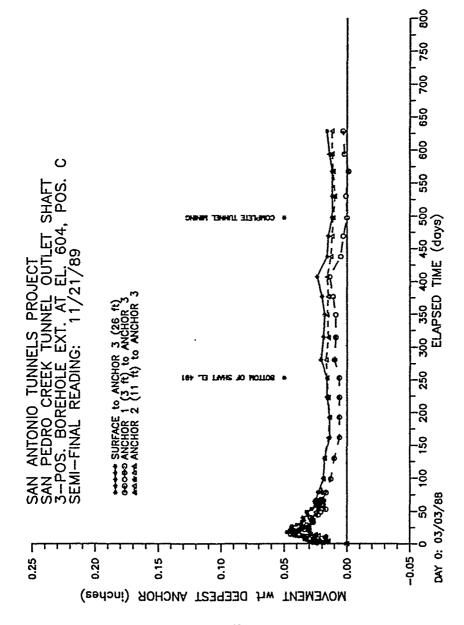
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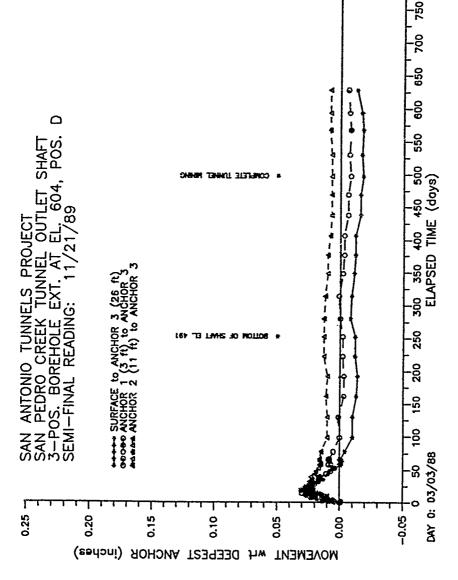
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OUTLET SHAFT



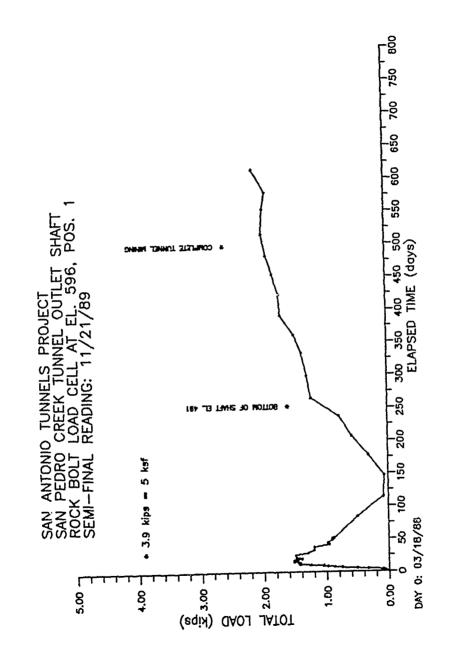


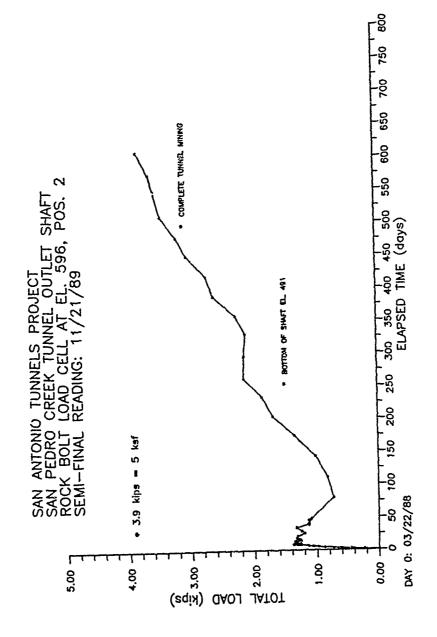


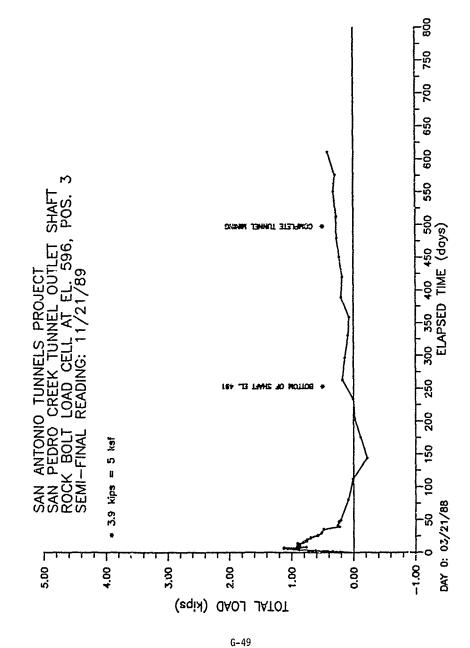


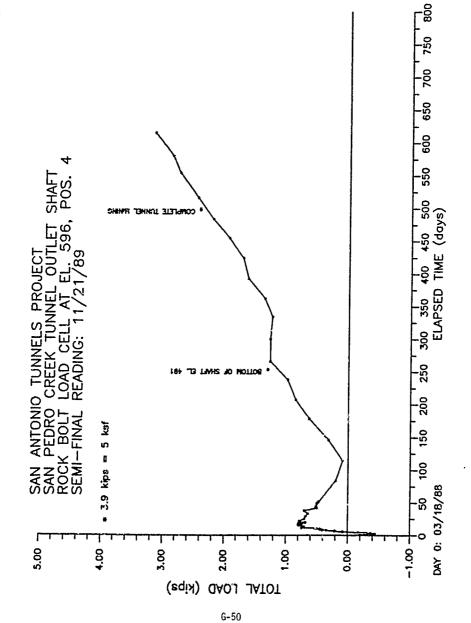
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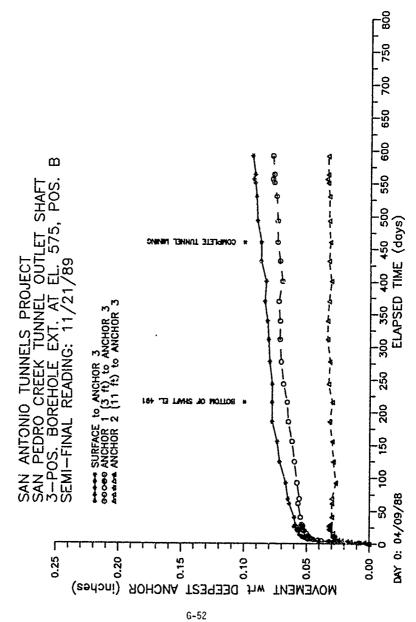
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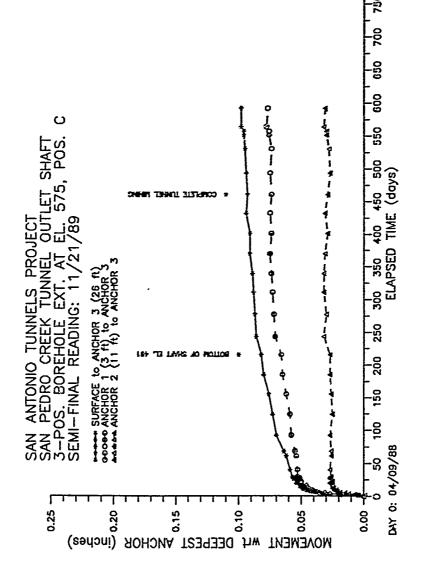


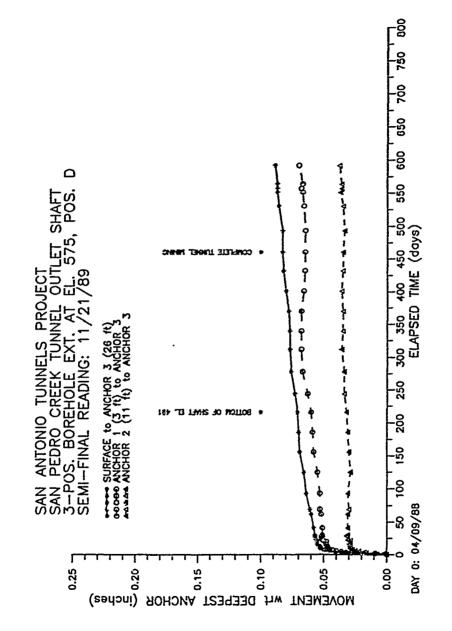


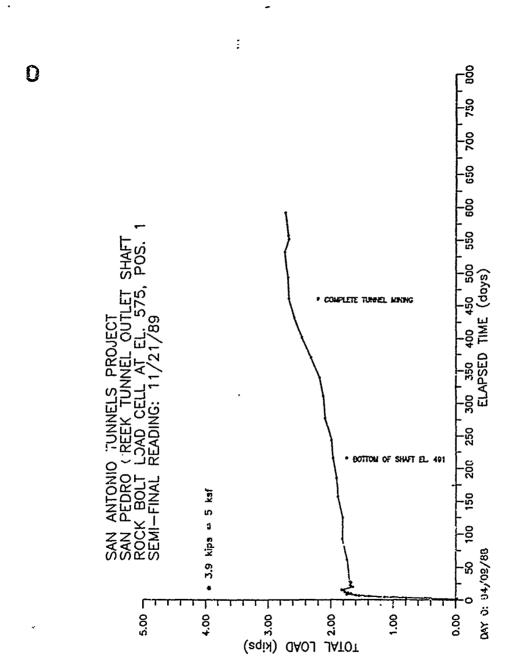




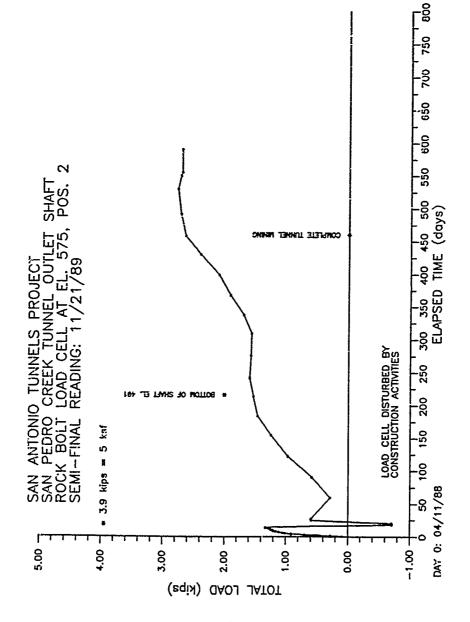


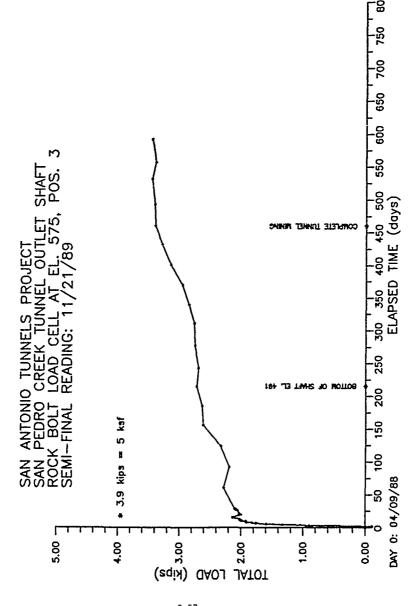




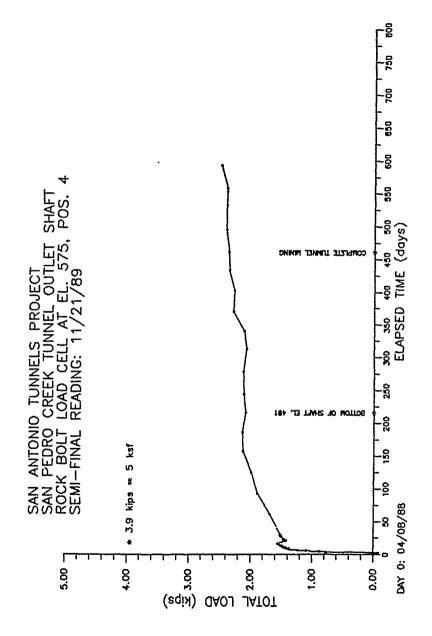


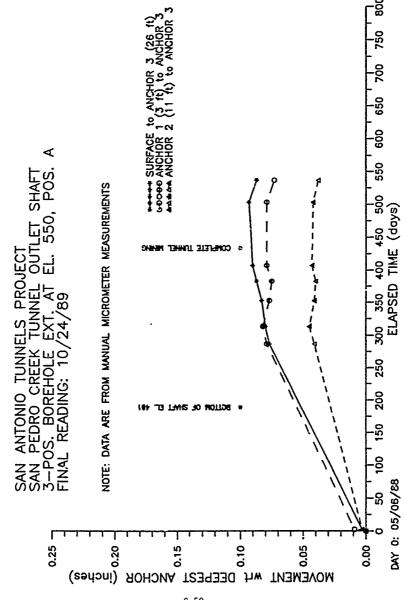


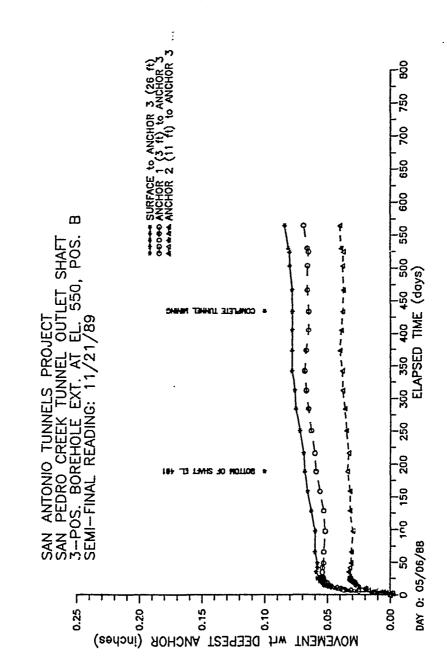


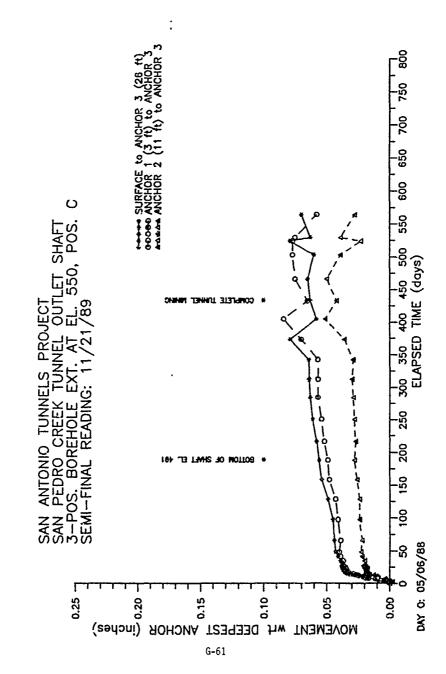


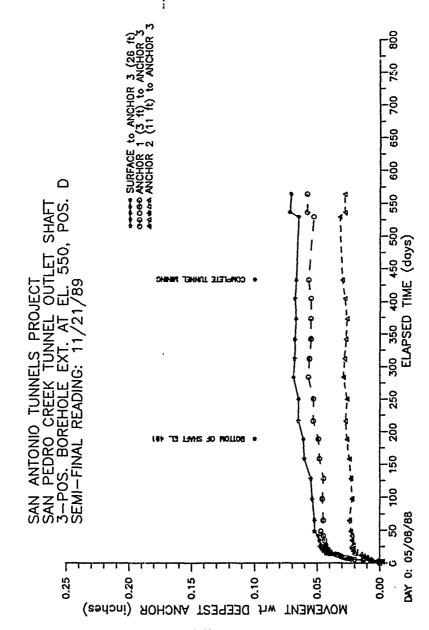


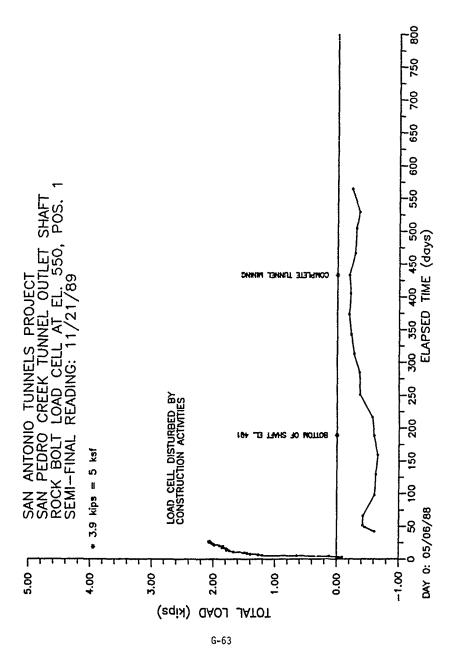




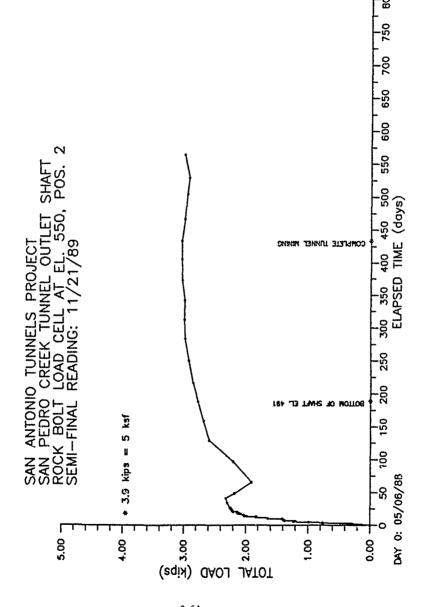


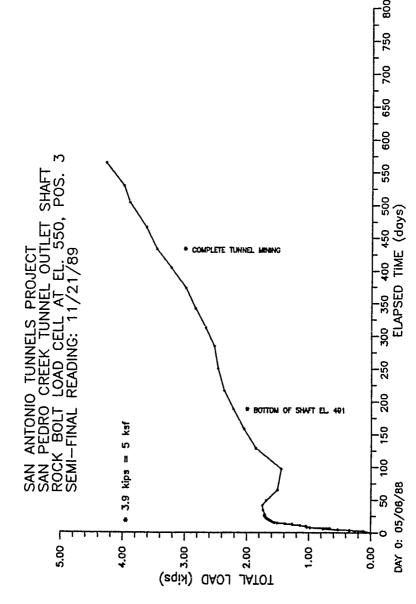


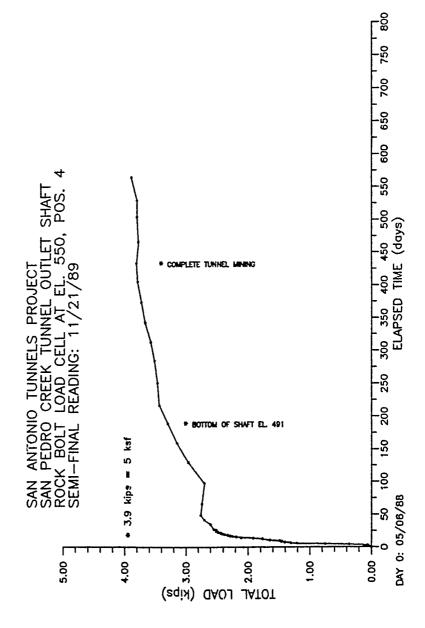




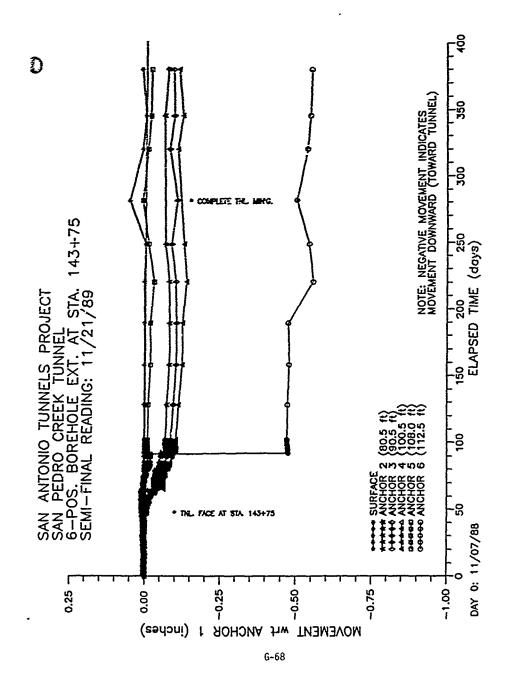


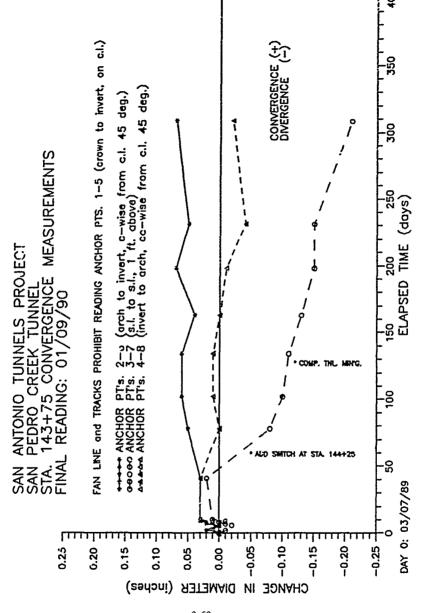


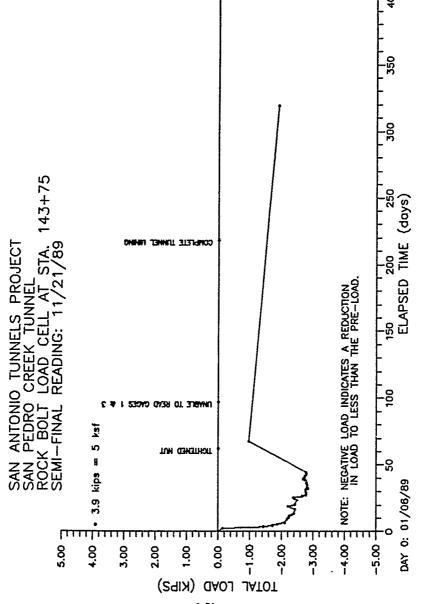




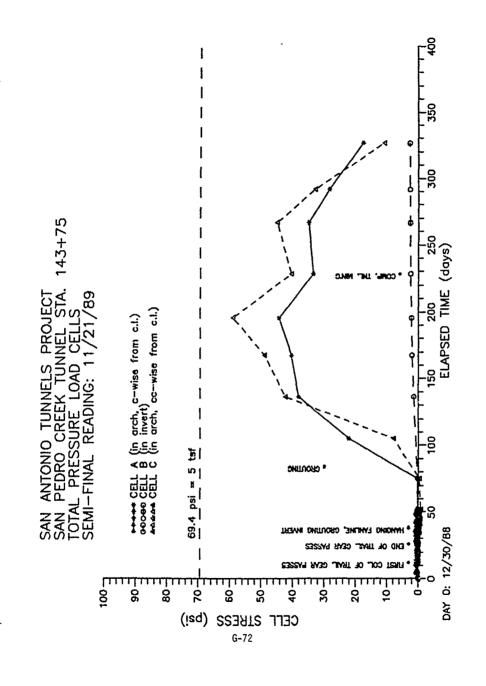
### TUNNEL STATION 143+75





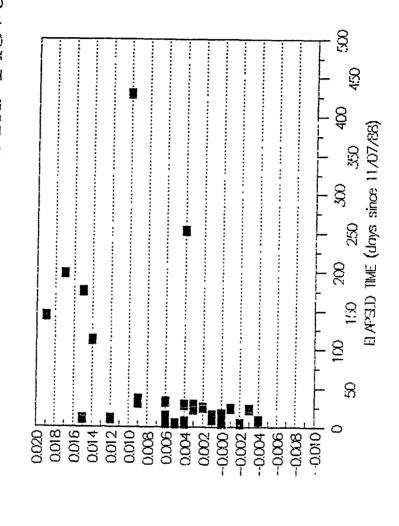


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## SAN PEDRO CREEK TUNNEL

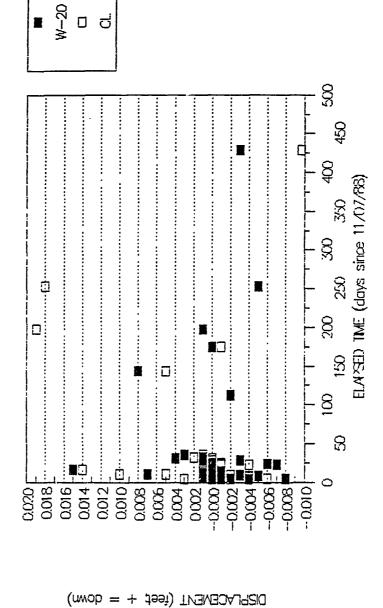
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DISPLACEMENT (feet + = down)

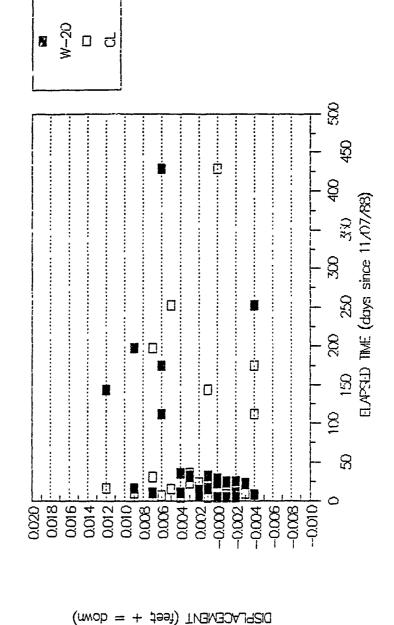
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# DISPLACEMENT MARKERS-STA. 143+05.5

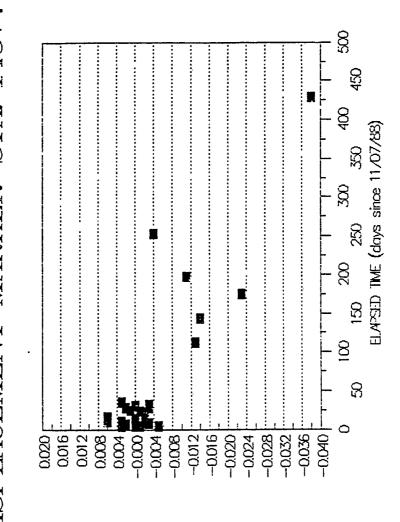


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### DISPLACEMENT MARKERS-STA. 143+40



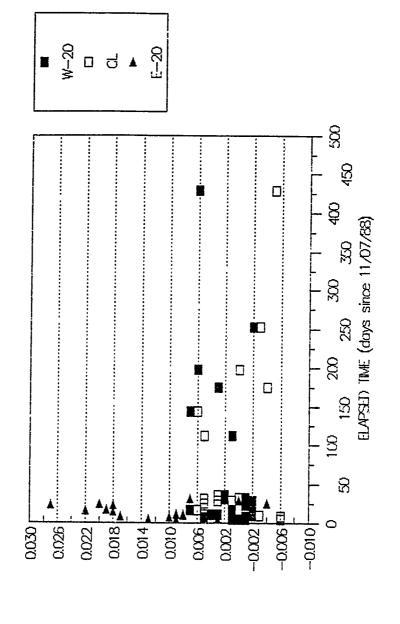
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ORPLACEMENT (feet + = down)

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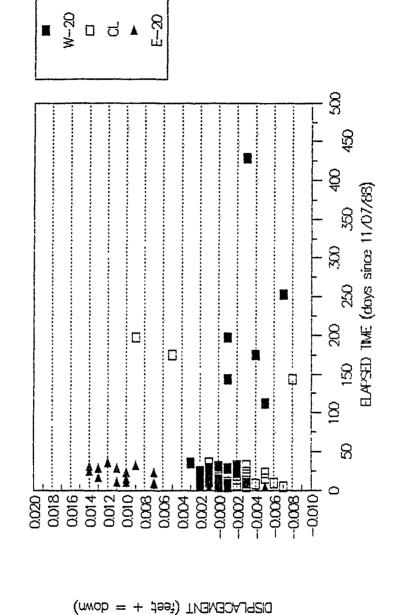
### DISPLACEMENT MARKERS-STA. 143+80



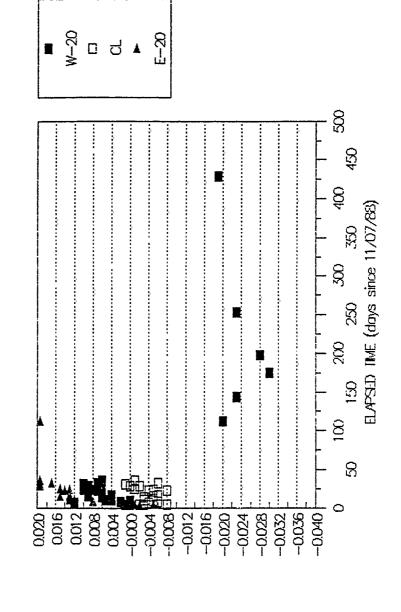
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### DISPLACEMENT MARKERS-STA, 144+20



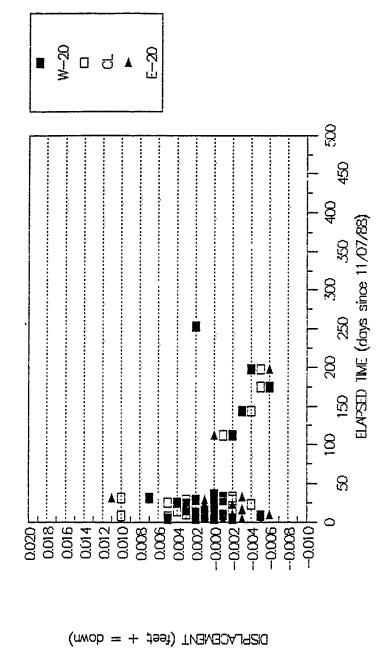
## DISPLACEMENT MARKERS-STA. 144+60



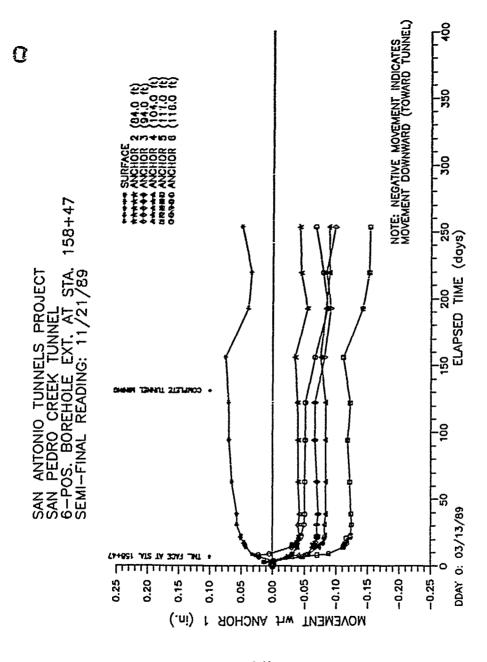
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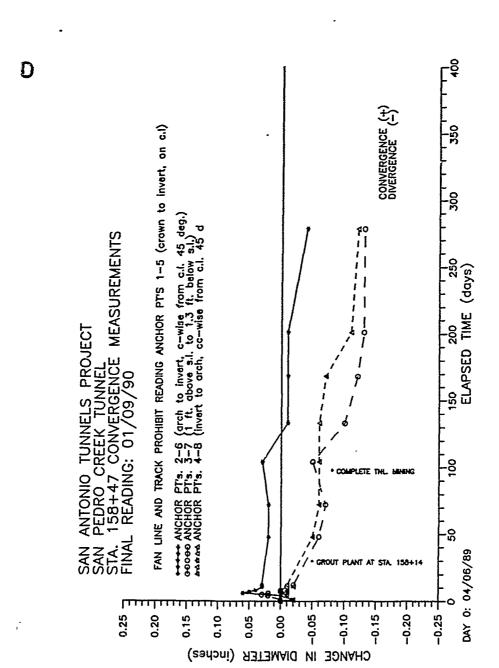
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## DISPLACEMENT MARKERS-STA, 145+00



TURNEL STATION 158+47

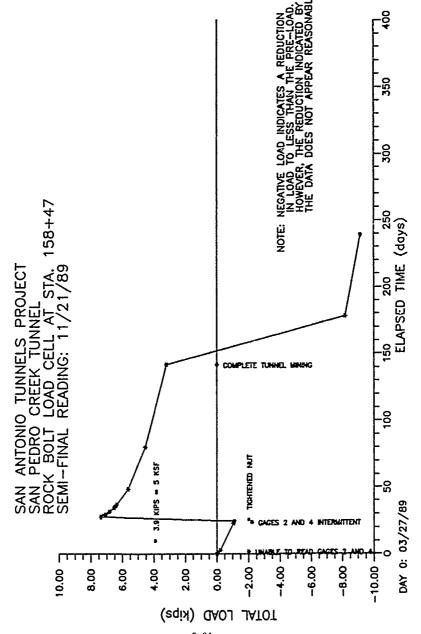


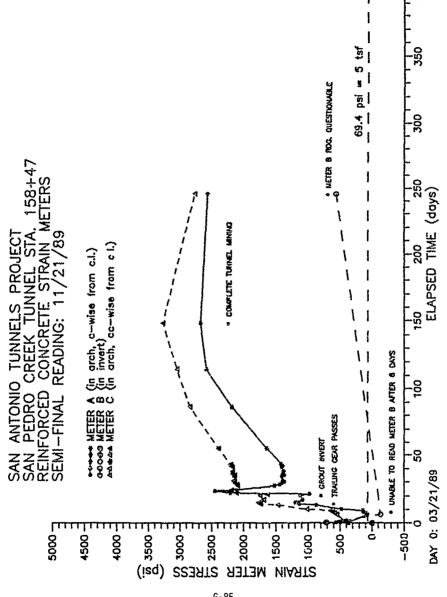


**DIAMETER** 

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